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CONTRIBUTIONS

FROM THE

SANITARY RESEARCH LABORATORY

AND

SEWAGE EXPERIMENT STATION

VOLUME IV

REPRINTED PAPERS

BOSTON, MASSACHUSETTS
1908

For Professor V. C. Vaughan.

With the Compliments of the

Sanitary Research Laboratory and Sewage Experiment Station of the Massachusetts Institute of Technology.

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INTRODUCTION: THE PRESENT STATUS OF THE SEWAGE-DISPOSAL PROBLEM. ACKNOWLEDGMENTS

By WILLIAM T. SEDGWICK, Director

WITH the appearance of a fourth volume of reprinted papers from a Sanitary Research Laboratory and Sewage Experiment Station originally endowed for three years only, the question will naturally be asked, Is the sewage-disposal problem moving towards a final solution? Or is this problem, like the problems of the origin of life, of the antiquity of man, of the age of the earth, and of the constitution of matter, simply another will-o'-the-wisp which recedes as fast and as certainly as it is approached?

To this question we may reply that the investigations of the last few years enable us to affirm without hesitation that real progress has actually been made, and practical results of far-reaching and permanent importance have already been reached; for whereas twenty-five years ago very little was known either about the nature of sewage or about its purification, and sewage-disposal was almost everywhere a purely empirical process, we have now reached a point where any community can deal with its sewage in at least one of several ways, according to its needs, and dispose of it at a reasonable cost; and this, too, without discharging upon the territory of its neighbors or into harbors, estuaries or other water courses more than a minimum of objectionable polluting materials.

The opening paper of Volume II (1905) of the present series consisted of a minute and careful survey by Professors Winslow and Phelps of all that had hitherto been done establishing or tending to establish upon a scientific foundation the theory and practice of sanitary and economical sewage-disposal. A large mass of material was digested, important conclusions were drawn as to the worth or worthlessness of various procedures, and some of the paths were indicated along which new and promising investigations needed to be made. No evidence was found of any tendency towards new difficulties in the way of sewage purification due to unforeseen characteristics of the municipal sewages of American cities, and no changes of condition of sewage in general, excepting those due to increasing concentration or arising from the wastes of special industries in particular places. The sewages of different cities, even in one State, differ indeed to an important extent, but yet as a rule not so widely as to affect qualitatively the methods available for their purification.

Since that time increasing experience has only confirmed and strengthened the same conclusion. Civilization advances after all but slowly; customs tend to become fixed and crystallized, and the sewages of cities, excepting as regards concentration, show no great or fundamental progressive changes in chemical, physical, or biological characteristics. Here clearly is no will-o'-the-wisp eluding our grasp because receding as we draw near, but on the contrary a problem which, as cities grow and industries become established and customs settled, tends everywhere to become more and more similar.

On the other hand, and at the other end, the achievements of the last quarter century, and especially of the last few years, in the science and art of sewage purification prove that remarkable progress has been made towards a complete and permanent mastery of the municipal sewage-disposal problem.

Reduced to its lowest terms this problem is simply: *how economically, and without entailing counter objections, to make unobjectionable infected streams of fluid filth.* Some elements of this perhaps seemingly simple problem were not at the outset sufficiently attended to, and counter difficulties arose; as, for example, in the method of disposal by chemical precipitation, by which process the sewage nuisance was partly overcome, but to a great extent replaced by the sludge nuisance. Under chemical precipitation streams of sewage discharged by municipalities were rendered comparatively unobjectionable to sight and smell, but the infections which they bore were not removed and a new and serious sludge nuisance was created. Again, intermittent filtration, which in many cases is still the best method of disposal, is for very large cities often uneconomical and impracticable because of its requirement of immense land areas of a special sandy or porous character.

As time has gone on it has gradually become more and more plain that the ideal sewage purification would be one which, while converting the filth borne by the stream into unobjectionable substances, should still keep these substances in solution or suspension, so that they should pass away with the stream and entail no nuisance from deposits; this change, of course, being effected at a tolerable and if possible at a small cost. These conditions the "trickling" or "percolating" or "sprinkling" filtration system promises better than any hitherto devised to fulfill. By this system the filth of a sewage stream is not largely deposited, or even removed, but only changed; so that, although in substance mostly retained, it is in character rendered unobjectionable.

But even this system fails in one important particular, namely, as regards disease germs. Trickling filters do not with any certainty remove disease germs, always possibly present in a sewage stream; and for this reason it becomes necessary in many cases to disinfect somehow the effluents of such filters. That such disinfection can be done without serious difficulty and at reasonable cost has been established by tests made at the Experiment Station of the Massachusetts Institute of Technology, and described in some of the papers contained in the present volume. The experiments here detailed, as well as those made by other investigators elsewhere, make it clear, moreover, that such disinfection can be done if required not only effectively, but also cheaply; as, for example, by the addition of a simple solution of bleaching powder.

These two processes then, namely, the transformation of the organic matters of sewage capable of decay into substances which will not readily rot, and the destruction of disease germs, are thus brought about, — the former by the utilization of a purely natural process, and the latter by simple, artificial means.

But in order to learn how to use these processes advantageously and economically, careful and extended experiments are still required both in general and sometimes also for particular sewages in particular places. The papers which follow describe such experimentation, of which the results already reached are highly satisfactory and encouraging.

By the continued generosity of the public-spirited Donor to whose munificence we are already so greatly indebted, further experiments are already well under way having for their end the ascertainment of the remaining practical data; and the next volume of the present series of "Contributions" will contain an account of such experiments, covering another year's work.

The staff of the Laboratory and Experiment Station remains, happily, unchanged, so that continuity of operation and long familiarity with the special problems involved, as well as scientific sequence in their treatment, are insured.

Whatever credit accrues from all these investigations belongs, first, to the Donor, who has incited and steadfastly supported them; second, to the Massachusetts Institute of Technology, which has administered and fostered them; and last, but not least, to Professors Winslow and Phelps, by means of whose personal devotion and technical skill they have been wisely planned and patiently executed.

DISPOSAL OF SEWAGE

BY

PROF. C. E. A. WINSLOW

MASSACHUSETTS INSTITUTE OF TECHNOLOGY
BOSTON, MASS.

Paper read at the Ninth Annual School for
Instruction of Health Officers
Burlington, Vt., June 19, 1907

Sewage disposal is a fundamental necessity because the presence of excreta means the existence of continual facilities for the transmission of disease. It is only necessary to study the statistics of communities, in which sewerage systems have been introduced, comparing typhoid death rates before and after that event, in order to realize how serious this danger is. In Berlin, for example, when a pure water supply was introduced the typhoid death rate dropped, but when a sewerage system followed some years later, another decrease in the death rate occurred almost as striking as the first. The reasons are many why sewerage systems thus diminish typhoid fever. The presence of privy vaults is a constant menace to wells and other private sources of supply; but entirely apart from any form of water pollution there are abundant ways in which infection passes from improperly handled excreta to adjacent food supplies. The experience of the United States army during the Spanish War furnished an impressive object lesson along this line. One out of every five of our volunteer soldiers came down with typhoid fever during the brief campaign, in shocking contrast with the almost perfect immunity secured by the Japanese in their scientific progress through Manchuria. This national disgrace was directly connected with improper care of excreta as brought out by the study of typhoid incidence on different regiments. In peace, typhoid fever slays its multitudes every year from the same neglect of the same precautions. Flies play an important part in its transmission as they forage back and forth from vault to larder; and the fingers of the users of unclean privies are rich sources of infection for themselves and others. These are unpleasant truths, but I have no doubt that citizens of the towns you represent are to-day in the position of the bird that fouls its own nest, without being conscious of the fact, and it is your duty to awake them to the truth, however unsavory it may be.

The first step in the campaign against typhoid fever is to secure a public source of pure and wholesome water, thus guarding that food supply which most easily carries infection to large numbers of persons. The second step, only less urgent than the first, is to so care for excreta that infection may not be spread broadcast through various other channels. In many communities this must be done by substituting for leaky, ramshackle privies earth closets of adequate construction. To prevent danger, it is necessary that excreta should be received in some receptacle which is water tight and closed against flies. Earth or ashes should be at hand to cover them at once, and provision must be made for burning or burying the contents at frequent intervals. Light and light's brother—cleanliness—are of course essential in the privy building itself.

The proper maintenance of such earth closets is unquestionably a burden, and anything less than this is a public danger. The cesspool, too, which marks the next step forward, is, after all, only a makeshift, since the average citizen has neither the inclination nor the ability to ensure its proper operation. As soon as possible, therefore, the introduction of a water carriage system of sewerage must follow with the growth of the sanitary conscience. This is the ideal method of disposal for the householder, since it removes all excretal matter promptly and efficiently and meets, without trouble on his part, all sanitary and esthetic requirements. It creates, of course, a new problem for the community as a whole; and it is of this problem that I wish chiefly to speak to you this morning. By the construction of a public sewerage system the problem of disposal is shifted one step away from the individual; instead of the individual having a small amount of waste to dispose of, the municipality has now a much larger problem on its hands—the disposal of all waste materials diluted with a great volume of water. For example, in Boston, on one side of the city, known as the South Metropolitan District, it is necessary in every year to dispose in some way of 1,600 tons of nitrogen as free ammonia alone, representing an enormous quantity of organic matter; and the problem is made so much more difficult by reason of the fact that the organic matter is diluted with thirty billion gallons of water.

In American sewages we find only one part in 1,000 of solid matter; in other words, there are 999 parts of water to be handled, or somewhat more than that, for every part of solid matter. The dangerous elements making up the single part in 1,000 with which we are particularly concerned in sewage disposal may be classed under three heads; organic matter, mineral matter and bacteria. The mineral matter is harmful only under exceptional conditions, when large amounts of industrial wastes enter the sewers and need not be considered here. The bacteria are harmful when the sewage is so discharged as to reach some source of food or water supply. The organic matter is harmful when it decays, and creates a nuisance which may be so serious as to menace health. Such organic matter is composed of unstable molecules, imperfectly oxidized, the history of which may follow two different lines. On the one hand, if it remains out of contact with the air, it may decompose, putrefy, break down, with the production of offensive gases of decomposition. Or, secondly, under the influence of oxygen it may undergo another process that we call nitrification, a slow burning or combustion which converts the organic matter into nitrates, or other mineral substances, without the production of foul odors, and in a wholly satisfactory way. Organic matter must either putrefy or nitrify, and the aim of sewage treatment is to nitrify it. This is a difficult problem, because it involves the supply to that organic matter of one to three times its weight of oxygen, with the special conditions under which the oxygen and the organic matter will unite.

The most obvious way to dispose of sewage is to throw it into the nearest body of water. Before true sewers existed the natural drains discharged into the nearest water source, and when these drains became filled with polluting matter the same plan was followed. Sometimes this works very well. If the volume of sewage discharged into a stream is sufficiently small, there may be enough oxygen to unite with the organic matter and enough organic life in the stream to effect the union. Under such condition there results a self-purification of a very satisfactory type, as far as the chemical constituents are concerned. Disease bacteria on the other hand, although they do die out in water, decrease slowly and uncertainly, so that bacterial self-purification cannot be relied on to make polluted water potable. Even chemical self-purification has its rather narrow limits. The fate of the sewage changes entirely when its volume exceeds the maximum which may be absorbed by a given stream, roughly stated as one part of sewage to fifty parts of water. When that limit is exceeded, when the limit of available oxygen for the oxidation of organic matter is passed, then the conditions of putrefaction are set up. It is like Mr. Micawber's philosophy: "Annual income twenty pounds, annual expenditures nineteen ought and six, result happiness. Annual income twenty pounds, annual expenditures twenty pounds ought and six, result misery." Up to a certain point everything goes well, but beyond that point, conditions are totally changed and instead of a self-purifying stream you find a foul septic tank. Dilution then—the discharge of sewage into water—is limited by the volume of sewage in relation to the body of water into which it is to discharge. On the seacoast the conditions for purification by dilution are ideal; and with large rivers like the Mississippi this method may be followed very satisfactorily. In smaller streams we get the septic tank condition of which I have spoken. In Massachusetts alone there are at least seven rivers so polluted as to become a public nuisance for a part of their course. Most serious of all, when such a condition as this is reached, there is often so deep a deposit of decomposing sludge in the bed of the stream that after the removal of all fresh pollution it remains as foul and unsightly as before. Nothing but expensive dredging operations will suffice to restore a stream so desecrated to its pristine purity. I beg you to take warning from our more thickly settled commonwealth and avoid while you may such problems as we now find before us in the Neponset and the Blackstone rivers.

A second obvious method of sewage disposal is discharge on the surface of the soil; and this plan has been followed from time immemorial. In the little city of Bunzlau in Prussia there was a public water supply as early as 1559, piped from a spring, and in connection with it was installed a sewerage system and a system of disposal on a sewage farm. Later the process of treatment on land underwent development along more intensive lines. As early as 1865 the German engineer, Mueller, urged the disposal of sewage through land at a more rapid rate than is possible on the ordinary sewage farm. He understood the nature of the process and knew that it was carried out by organic life; and in 1878 he patented a system "for the disinfection, purification and utilization of sewage by the scientific cultivation of yeast-like organisms." In 1870 the subject was approached along a different line by the English experts of the Rivers Pollution Commission under the leadership of Frankland. These experimenters failed to grasp the biological character of the process, but they understood its chemistry and showed that sewage could be successfully purified by filtration through various soils, at much higher rates than those used on sewage farms. It was only by work in this country, however, that these various lines of investigation were brought together in so conclusive a form that the importance of the process of intermittent filtration through sand was generally understood. The difficulties experienced by Worcester through the pollution by its sewage of the Blackstone River and the proposition to treat the sewage of Boston and the cities lying to the north, on the Saugus marshes, made sewage disposal an urgent question in Massachusetts in the early eighties. As a result of these conditions the Lawrence Experiment Station of the State Board of Health was founded in 1887 for the study of the sewage purification problem. There, under the leadership of Mr. Hiram F. Mills, with the coöperation of Prof. T. M. Drown and Prof. W. T. Sedgwick, were carried out the series of experiments since sightly spoken of as classic. Ten circular cypress tanks seventeen feet in diameter and six feet deep were filled with various filtering materials—sand, gravel, peat, river salt, loam, garden soil, and clay. The fine materials like peat, garden soil, and clay soon clogged, but the beds of sand worked to admiration, receiving sewage above and yielding a clear, bright spring water below. Some of the original tanks have been operated from the day of their installation to the present, a period of over seventeen years, with continuously good results.

The Massachusetts investigators established very clearly that the purification was an oxidizing process, carried out by bacteria living in the sand, and that a rich supply of oxygen was necessary for their activity. In order to supply the oxygen it was necessary to dose the filters intermittently, putting on perhaps one dose a day in an amount of some 60,000 gallons per acre. These experiments led at once to the installation of a number of intermittent sewage filters in Massachusetts, and in 1903 twenty-three were in operation in different parts of the state, the best known being those at Marlboro, Brockton, Framingham, Gardner, and Clinton; some of them have been in operation now for over ten years. Where glacial drift sand or lake or river sand is available, the construction of such filters is extremely simple. It is necessary only to remove the soil and subsoil, which may be used for embankments between the individual beds; to dig up the sand at intervals for the laying of underdrains to carry off the effluent; and to level the surface of the beds again and place upon them some simple form of wooden carrier to secure a fairly even distribution of the sewage. Some arrangement must also be made for discharging the sewage on the beds, one dose of 50,000 gallons, the sewage of 500 persons, or two doses of half this amount being applied to each bed once in twenty-four hours. This may be done by hand or accomplished by one of a number of very satisfactory automatic devices now on the market.

In the operation of such beds it has been found that a certain amount of solid material accumulates on the surface, but this sledge is of a stable nature, not subject to decay, and can be scraped off at intervals and removed without excessive expense. Cold winter weather interferes somewhat with the efficiency of the process; but by ridging the beds so that the sewage may flow in furrows under the ice, it is possible with most of the filters in Massachusetts to treat the sewage all the year round.

In districts where it is difficult to secure adequate natural sand beds and where the construction of artificial ones would prove expensive, it is important to increase the rate at which sewage may be purified, and this may be done by the combination with intermittent filtration of preliminary treatment in the septic tank. This device is the logical development of the cesspool and the fosses fixées, used in France by Mouras as early as 1860. The principle—that the storing of sewage without access of air led to a sort of fermentation, or ripening, which removed a considerable portion of the solid material—was patented in 1882. Scott-Moncrieff an English engineer, in 1891 used the same process in what he called a cultivation tank, and in 1894 one was installed at Urbana, Ill., by Professor Talbot. The process did not attain wide popularity, however, until after 1895, when Donald Cameron of Exeter, England, gave it the picturesque name of the septic tank. It must be remembered that this scientifically controlled cesspool, as it really is, does not purify the sewage in any real sense. Only oxidation does that, and oxidation requires air. This anerobic fermentation (fermentation without air) serves simply to change solid material to the liquid or gaseous form and thus facilitates its ultimate purification and makes it possible to treat more sewage on an acre of land. Cameron in his early experiments used a tightly closed tank, but he soon found that this was needless; it was only necessary to allow the liquid to flow slowly through any tank, so as to have a storage of about twenty-four hours, the exact length of time varying with different sewages. Under such conditions decompositions take place, which remove from 60 to 70 per cent of the suspended solids, turning a part into liquid form, giving off a part as gas, about 7½ gallons of gas to 100 gallons of sewage, and storing a certain amount of solids in the tank. The proportion of the solids destroyed in the tank—digested, so to speak—varies with different sewages and with the same sewage, at different times, under conditions not thoroughly understood. In some cases the use of a septic tank is much more advantageous than in others. Small towns, however, where the sewage arrives in a comparatively fresh condition at the disposal area, will almost always find it beneficial.

The septic tank and the intermittent filter will generally form the most suitable combination for treating the sewage of a New England town. For small communities or institutions or large isolated country houses a modification of this process may be used which obviates the discharge of sewage upon the surface of the ground—an important esthetic factor under some conditions. This is the method of sub-surface irrigation in which the effluent from the septic tank is discharged through a system of open-jointed tile pipe laid a few inches below the surface of a sandy soil. The principle is the same as that of intermittent filtration. Since the sewage is not in free contact with the air, the process of nitrification is more imperfect and the pipes are liable to clog. In many cases, however, the freedom from the somewhat unsightly surface disposal area outweighs these objections. For small installations a plant of this type may be built very cheaply indeed. Mr. John W. Alvord of Chicago has designed such a plant consisting of a barrel for a septic tank, a half barrel equipped with a simple siphon for discharging the intermittent dose, and a system of tile drainage, all under ground, and costing for labor and materials less than twenty-five dollars.

In rocky and clayey districts even the addition of the septic tank will not serve to bring processes of purification through sand within reasonable limits of cost. In England, particularly, the problem has pressed hard; and there progress has been made along a different line. The chemist of the London County Council, Mr. W. J. Dibdin, undertook in the early nineties to devise some more rapid method of sewage disposal than that worked out at Lawrence. He increased the rate by the use of coarse material, following out the principle that the essential element in purification is not really filtration, but the action of micro-organisms which will grow on broken stone as well as on sand. With such coarse material, the sewage must somehow be retained sufficiently long for the bacteria to act on it, and this end Dibdin attained by making his bed (filled with coke) tight, so that he could fill it with sewage, allow it to stand for a certain period and then empty it. This he called the contact system because the sewage is held in contact with the coke by the closing of the outlet. In further experiments at Sutton in Surrey, Dibdin developed the double contact system, carrying the sewage treated in one contact bed to another contact bed and treating it again. By each contact about half the impurities in the applied liquid are removed. It has been found that in the construction of the contact beds almost any hard, smooth material may be used, the bed being simply a tight box for holding it. Coal and coke are particularly favorable, though stone or almost any other hard substance may be used. The size of the pieces of material, whatever it may be, varies from ⅛ of an inch to 1½ inches. The beds are commonly made from three to six feet deep, or thereabouts, but the depth does not seem to be of great importance. The sewage is run slowly into the bed, allowed to stand for a certain time, generally two hours, and then discharged, three or four fillings being made in the twenty-four hours. With such a system sewage may be treated at five or ten times the rate attainable by sand filtration and the effluent, though never purified to the grade of a good spring water, as the effluent from an intermittent filter may be, is so stable that it will not decay and may be discharged into a river with safety.

A still more satisfactory method by which sewage may be treated at even higher rates is the process of trickling filtration, also devised in England. This process secures the necessary even distribution and aeration by sprinkling the sewage over the whole surface of the filter bed in a fine spray, so that the oxygen goes in along with it, the sewage never filling the tank, but continuously trickling through the filtering material. One of the advantages of the trickling bed system is the extreme simplicity of its construction. Some kind of floor sloped to carry off the drainage is needed, but aside from this the material used can be held together by wooden palings or iron bands or any very simple structure around the outside. The depth is generally four feet or over, and as in the case of the contact bed, any hard material may be used, varying in size from ⅛ inch to 1½ inches.

A serious difficulty arises in regard to the distribution of the sewage, which must be so applied as to pass through slowly and well mixed with air. The plans for filters at Columbus, Ohio, like the latest English designs at Birmingham, call for fixed sprinkler heads which throw the sewage up into the air in a fine spray. This device is a good one and may be much improved by the addition of siphon tanks which intermit the discharge at frequent intervals so as to expand and contract the cone of spray and spread the sewage evenly. In the course of our experiments at the Technology Sewage Experiment Station, we have devised another method of distribution which may offer especial advantages in certain cases. This consists of a system of troughs from holes in the bottom of which the sewage drops upon concave discs from which it splashes upward in a fine and well spread spray. This system of gravity distribution was described in the *Engineering News* a year ago and has since been adopted by the Massachusetts State Board of Health for the experimental sewage filter at Andover.

With good distribution, the rates which may be obtained by the trickling process are very high. The intermittent sand filter will take at the outside 100,000 gallons, and the contact bed 500,000 or 600,000 gallons per acre per day; the trickling filter, on the other hand, easily treats 2,000,000 gallons per acre per days, giving an effluent on the whole better than the contact bed, though not so good as the sand filter. It contains a considerable amount of suspended solids and does not look as clear as the effluent from the contact bed. But it analyzes well and keeps well, which are the principal points to be considered. Furthermore, its solids may be easily removed by a short period of sedimentation.

Returning to the distinction originally made between the chemical and bacterial constituents of sewage, it must be remembered that these rapid processes affect only the organic matter. They are essentially oxidizing mechanisms without filtering action adequate for the removal of microorganisms. It is true that in the unfavorable environment of the septic tank and trickling filter, many sewage organisms do die out, but their elimination is incomplete and uncertain. If a nearly germ-free effluent is desired, as in cases where sewage works are situated upon estuaries where shellfish are cultivated, special means must be taken to remove bacteria. This may be done by chemical treatment, and the development of methods to this end marks the newest step in the growing art of sewage disposal. Chemicals of various sorts were tested most elaborately as precipitants of sewage in the early seventies with poor results. As a purifying process, for the removal of organic matter, chemical treatment is of dubious expediency since the soluble constituents are imperfectly removed and the cost of the process, with its attendant necessity for sludge removal, is generally far greater than the expense of more adequate biological treatment. When organic stability has been attained, however, as in the effluents from rapid processes, chemical treatment may be used with marked success for the purpose of destroying bacteria. Experiments carried out by my colleague, Professor Phelps at the Technology Experiment Station, have demonstrated beyond doubt the cheapness and efficiency of chloride of lime for this particular purpose. Such doses of the chemical as are economically practicable do not destroy all germ life, but they do kill all ordinary sewage and pathogenic organisms, leaving only a few harmless spore formers. The process is not therefore sterilization, but it is disinfection. I am myself inclined to believe that this procedure may not only find application in the treatment of trickling and contact effluents, but in some cases may be used for crude sewage as well. Many towns turn their sewage into lakes or large streams competent to handle for a long time to come all the organic matter which may be discharged into them. In some such cases, the pathogenic germs present do constitute a menace to other communities or to the community chiefly concerned. Disinfection offers a promising method for securing at least temporary relief from such a difficulty.

The particular scheme of treatment to be adopted for any community can only be intelligently selected after a careful study of local conditions. Columbus, Ohio, is the first American city—the Lawrence station being, of course, a state institution—to approach the subject of sewage purification in a rational way. About three years ago the city called in Messrs. Hering and Fuller to investigate and report on the best method of sewage treatment, and appropriated nearly fifty thousand dollars for experimental work. A fine experiment station was equipped in which studies were made of over forty separate methods of disposal. The experts have finally reached the conclusion that the best treatment for Columbus sewage is by the use of the septic tank, followed by the trickling filter, and this again by sedimentation; and they are now proceeding with the construction of the largest plant of this type on our side of the water, including ten acres of trickling filters. Baltimore, Md., is now carrying out investigations of a still more extensive character, along generally similar lines.

The sewage experiment station of the Massachusetts Institute of Technology differs from any other similar testing station in the fact that its scope is wider than a particular local problem. Its purpose, in accord with the wishes of its anonymous founder are to advance the science of sewage disposal in both theoretical and practical aspects and to promote popular knowledge of the nature and importance of this important field of sanitary work. The purely scientific chemical and bacteriological questions involved have naturally claimed a large sphere of our time. We have thought it wise, however, to investigate also the practical problem of the treatment of Boston sewage, now discharged unpurified into the harbor, but likely at some future day to require more careful handling. As at Columbus, the trickling filter seems to offer the best solution of the problem, preceded by septic treatment.

There are many problems still to be solved in the purification of sewage. The removal of suspended matter and its digestion in the septic tank, for example, urgently demand a careful study; yet the work of the last ten years in England and the United States has blocked out the main outlines of satisfactory sewage disposal practice. The engineer can to-day successfully meet any demand for the purification of domestic sewage; and this purification may be carried to any degree of perfection for which the community in question is prepared to pay. If you want a clear and sparkling effluent, highly purified bacterially, he can design you an intermittent filter for that purpose. If you merely want a stable effluent which may be discharged into a stream without creating a nuisance, he can build you a trickling filter. If you want, on the other hand, a disinfected but not organically purified effluent, that end, too, may be attained.

In closing, let me add that what Mr. Fuller said about water purification is equally true of sewage disposal—a badly constructed or badly operated system is worse than none at all, for it creates a false sense of security and blocks the path of true progress. When your community reaches the point when sewage disposal should be attempted, have your local situation studied and your plant carefully designed and place it under conscientious and intelligent supervision. A sewage disposal project so carried out will prove one of the soundest investments any community can make; for its dividends will be paid by the saving of human lives.

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Contribution from the
Sanitary Research Laboratory and Sewage Experiment Station
of the Massachusetts Institute of Technology

INVESTIGATIONS ON THE PURIFICATION OF BOSTON SEWAGE IN SEPTIC TANKS AND TRICKLING FILTERS (1905-1907)

By C.-E. A. WINSLOW and EARLE B. PHELPS

I. Results of Previous Investigations

The Sanitary Research Laboratory and Sewage Experiment Station of the Massachusetts Institute of Technology was founded in 1902 by an anonymous donor for the purpose of making experiments upon improved methods of sewage disposal, especially those adapted to large cities. A large part of the work of the staff has been devoted to the study of the more purely scientific problems — chemical, bacteriological, and hydraulic — which underlie the practice of sewage analysis and sewage purification. We have carried on, however, along with these theoretical investigations, an experimental study of the immediate local problem of sewage disposal, as it is certain some day to confront the city of Boston.

The sewage of the Metropolitan district of Boston is at present discharged, untreated, into the waters of the harbor at three different points. The main outfall sewer of the north district carries some fifty million gallons daily, and discharges continuously off Deer Island. The sewage from the high level district passes out to Peddock's Island, near the southeastern limit of the harbor; this amounts to about twenty million gallons. The main outfall of the south Metropolitan district is at Moon Island, nearer the centre of the harbor, and here the sewage is stored in masonry tanks and discharged only on the turn of the tide. The daily flow in this sewer is in the neighborhood of one hundred million gallons. The Massachusetts State Board of Health investigated the condition of Boston Harbor in 1905 (Goodnough, 1906), and found no serious damage from this method of disposal. The town of Wellesley has, however, recently been refused admittance to the Metropolitan system from fear of overtaxing the purifying power of the harbor (Wellesley, 1907);[1] and it can scarcely be doubted that the progressive increase of population within the drainage district itself will ultimately bring the problem of sewage purification to the fore. The present agitation in regard to the pollution of New York Harbor (Whipple, G. C., 1907) under somewhat similar conditions is an indication of what must some day be expected in Boston.

Under these circumstances it is important to form a general idea beforehand of what a proper purification of Boston sewage will involve; and the investigations at the Sanitary Research Laboratory have reached a point at which this can approximately be determined. It should be understood that neither state nor municipal authorities are responsible for the conclusions which follow. The investigations here reported proceed wholly from the Sewage Experiment Station of the Institute, and deal with a coming problem, not a present one. It is hoped, however, that they may yield data of importance in determining the feasibility of a disposal project when the time for installing one shall come, and offer some guidance as to its probable final form.

The first essential in planning for sewage disposal is a knowledge of the special characteristics of the sewage in question. In a previous communication (Winslow and Phelps, 1905) we have discussed this point somewhat fully. The sewage of the south Metropolitan district, with which we dealt in our experiments, is that which flows from the combined system of the city of Boston proper. It is a typical American domestic sewage, of average strength, and in 1903 morning samples showed an average of about six parts per million of nitrogen as albuminoid ammonia, eighteen parts of nitrogen as free ammonia, and forty-five parts of "oxygen consumed." Suspended solids were at this time somewhat under 150 parts per million. We have pointed out in the paper above cited that the average analysis of the whole twenty-four hours' flow was about 80 per cent. as strong as the morning sewage.

A preliminary series of investigations covering the various methods of purification in common use was carried out from 1903 to 1905. Septic tanks, contact beds, sand filters, and trickling filters were studied, each unit being a cypress tank of 72 to 96 cubic feet capacity, and the results have been fully presented in a previous communication

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[1] It is understood that this action was taken in view of the fact that Wellesley lies outside the area previously fixed upon as properly tributary to the Boston outfall.

(Winslow and Phelps, 1906). These studies led to certain general conclusions which may be summarized as follows:

It will be necessary in any treatment to screen out large floating bodies and to settle out mineral detritus. This process should be limited to a sedimentation of a few minutes. Under such conditions the settled material amounts to about 16,000 pounds (.65 cubic yard) per million gallons of sewage, and is of such a character that it may be spread out on land without fear of nuisance.

A further removal of the suspended organic matter may be effected, if desired, by treatment in the septic tank. In our experiments an open tank operated as well as a closed tank. Varying the storage period from twelve to forty-eight hours produced no difference in the effluents which was measurable by analytical results. The tanks studied removed nearly two-thirds of the suspended matter and yielded an effluent which was clear, but much darkened by sulphides. The tanks on an average received fifty pounds of nitrogen as albuminoid ammonia (dissolved and suspended), of which thirty pounds were discharged in the effluent, fifteen to seventeen pounds decomposed, and three to five pounds stored as sludge. In the decomposition of sludge the length of the septic period was of great importance. The amount of organic solids stored, undecomposed, per million gallons of sewage passed was twice as great with a forty-eight-hour period as with a twenty-four-hour period and four times as great as with a twelve-hour period.

Under the conditions of these experiments, crude Boston sewage was successfully filtered through a 2-foot bed of sand with an effective size of .14 mm., at a rate of .4 million gallons (400,000 gallons) per acre per day, divided into four doses in the twenty-four hours. Such high rates should not be expected in actual practice, with outdoor beds, particularly during the winter months. In communities with limited sand areas much might be accomplished, however, by frequent cleaning of the surface of the beds; and it is interesting to know that with care in construction and operation the sand filter may be efficient at higher rates than have been generally advocated. The effluents obtained from the sand beds in these experiments were clear, bright, and well purified. Preliminary septic treatment for twelve or twenty-four hours did not improve the effluents obtained with sand filtration, although it made the care of the surface of the beds somewhat easier.

It was found that crude Boston sewage could be treated in single-contact beds of fine stone (½ inch in diameter) at a rate of about

1.2 million gallons per acre per day. The effluent, though only partially purified, was generally so stable that it could be discharged into a considerable volume of water without any tendency to create a nuisance. The beds clogged rapidly and the surface needed much attention. The double-contact system of treatment, in primary beds of 2-inch material and secondary beds of ½-inch material, yielded a fairly well purified and stable effluent at a rate, on the combined double system, of about 0.7 million gallons per acre per day, with beds 6 feet deep. This system clogged much less seriously, but nevertheless lost sufficient capacity to require renewal every few years. Preliminary septic treatment obviated this loss of capacity to a considerable extent.

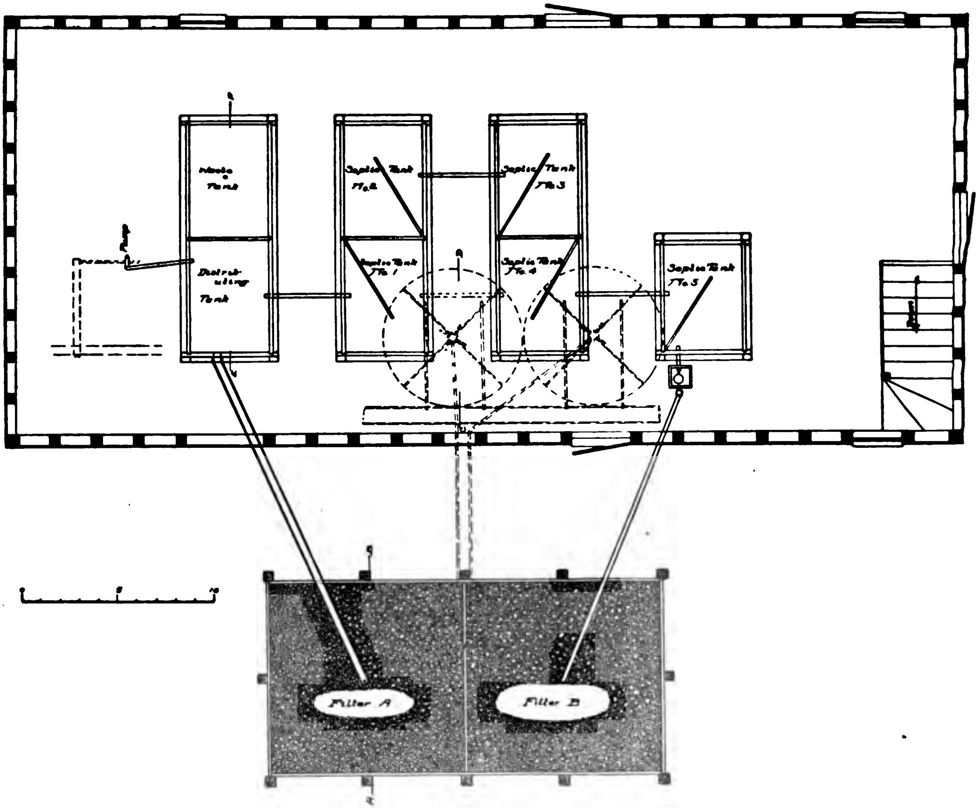


FIG. 1.—PLAN OF EXPERIMENTAL PLANT

Experiments with continuous, trickling, or sprinkling filters showed results equal to those obtained with the double-contact system at much higher rates, and suggested that this method would probably prove the most practical of all. Three such tanks were studied, one from 1903 on and the other two in 1904 and 1905. They were operated at rates between 1.5 and 2.5 million gallons per acre, producing turbid but fairly stable effluents, and were entirely free from surface clogging. To base conclusions as to the efficiency of trickling filters upon tanks only 16 square feet in area, dosed with tipping buckets and operated under cover, seemed, however, unwise. For such a general survey as was contemplated in our preliminary work, and particularly for studies of contact beds, small indoor filters are of value; but the two important points in the operation of the trickling filter are the system of distribution and the effect of winter conditions, neither of which can be studied on a small scale. We therefore constructed in August, 1905, an outdoor filter of adequate size, which has been successfully operated for nearly two years; this has yielded results which we believe to be representative.

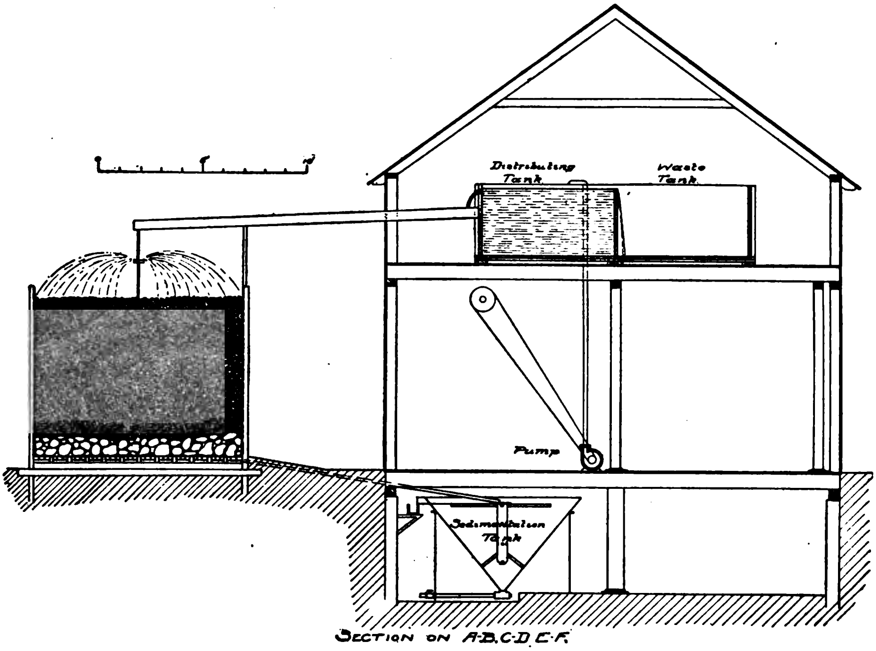


FIG. 2.—SECTIONAL ELEVATION OF EXPERIMENTAL PLANT

II. DESCRIPTION OF THE EXPERIMENTAL PLANT

The Sewage Experiment Station of the Massachusetts Institute of Technology is situated in the southeastern portion of the city of Boston, near the corner of Albany Street and Massachusetts Avenue. It includes a small, two-story laboratory building, a two-story tank house, and the trickling filter. Septic tanks and sedimentation basins are placed inside the tank house. The general arrangement of the tank house and filter is shown in Figures 1 and 2.

Sewage for the use of the station is obtained from the 9-foot trunk sewer of the Boston Main Drainage Works at a point where there is a flow of some fifty million gallons a day from a contributing population of 250,000. It is raised by a 4 × 6 Warren duplex pump, being roughly settled on the way by passage through a grit chamber, 19 inches in diameter and 16 inches deep, fitted with a screen whose bars are ½ inch apart. The sewage is then raised, without further sedimentation, by a ¾-inch centrifugal pump to the distributing tank, shown in the plan and section, Figures 1 and 2 No sludge was removed from this tank, and the inflow was kept always in excess of the outflow, a constant head being maintained by wasting over a 36-inch weir at one end. From the opposite end of the distributing tank sewage flowed to one of the trickling beds, filter A, over a 1-inch brass weir, the crest of which could be raised or lowered by a micrometer screw. By regulation of this crest in relation to the constant height on the waste weir it was possible to obtain any desired flow on the filters. A depth of about 1 inch on the small weir corresponded to 200 gallons per hour, or 2,000,000 gallons per acre per day, the rate in use for the past two years.

From one side of the distributing tank sewage flowed continuously through a short length of 2½-inch iron pipe to the first of the septic tanks. These were open cypress tanks, 6 × 4 feet × 3 feet deep, connected in series by 2½-inch iron pipe.

The general arrangement of the septic tanks is indicated in Figure 1. Only Nos. 1, 4, and 5 were used during the first twelve months of the investigation. In October, 1906, the direct connection between 1 and 4, and 4 and 5, was cut off and Nos. 2 and 3 brought into the series. The distributing tank and all five septic tanks have since that time been in open connection, the level in all being controlled by the large waste weir of the distributing tank. The septic tanks were provided with wooden baffles, arranged as shown in Figure 1. The total capacity of the septic system was equal to twelve hours' flow, and its division into five separate tanks made it possible to study successive stages in the septic process. From the last tank, a 1-inch brass weir, exactly like that on the distributing tank, regulated the flow on the second trickling bed, filter B.

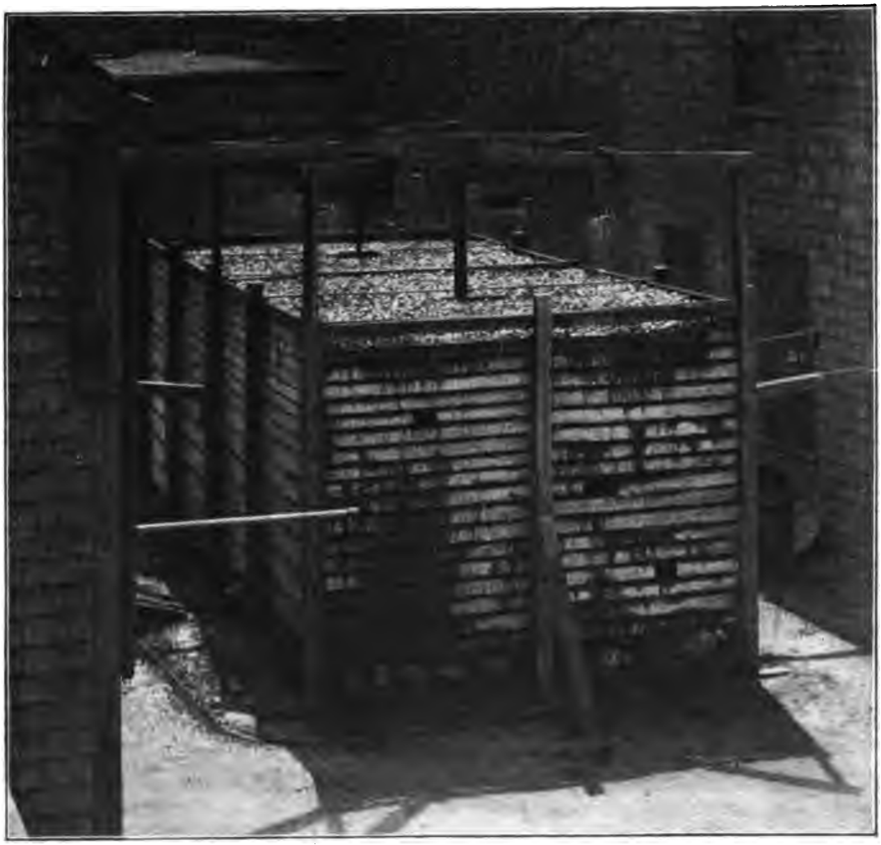


FIG. 3.—VIEW OF EXPERIMENTAL TRICKLING BEDS

A general view of the trickling beds is shown in Figure 3. The combined superficial area of both beds was 200 square feet, and the depth 8 feet. A central partition of 2-inch spruce planking, with matched joints, divided the total area into two separate beds, each 10 feet square. Filter A treated crude sewage and filter B septic effluent. Both beds were filled with crushed stone, granite, and trap, between 1½ and 2 inches in diameter.

The construction of the filters was as follows: For a foundation, 4 × 4 inch spruce beams were buried in the ground, with top surfaces flush. A hemlock floor 1 inch thick was next laid and covered with a layer of 1 to 2 Portland cement mortar, 1 inch thick at the inner corner of the filters near the tank house, and sloping up to a thickness of 3 inches at the opposite side. Four radiating channels in the cement, ½ inch deep, aided the flow to the inner corners, from which the effluent was conducted into the tank house by 1½-inch iron pipes, 8 feet long, with a slope of about 1 foot. The sides of the filter (Fig. 3) were of spruce planks, with ½ to 2-inch openings between them, the lower 18 inches being entirely open. These sides were supported by 4 × 4 inch uprights, braced on all sides and tied at the tops by ½-inch iron rods. On the cement floor two layers of brick were laid in open

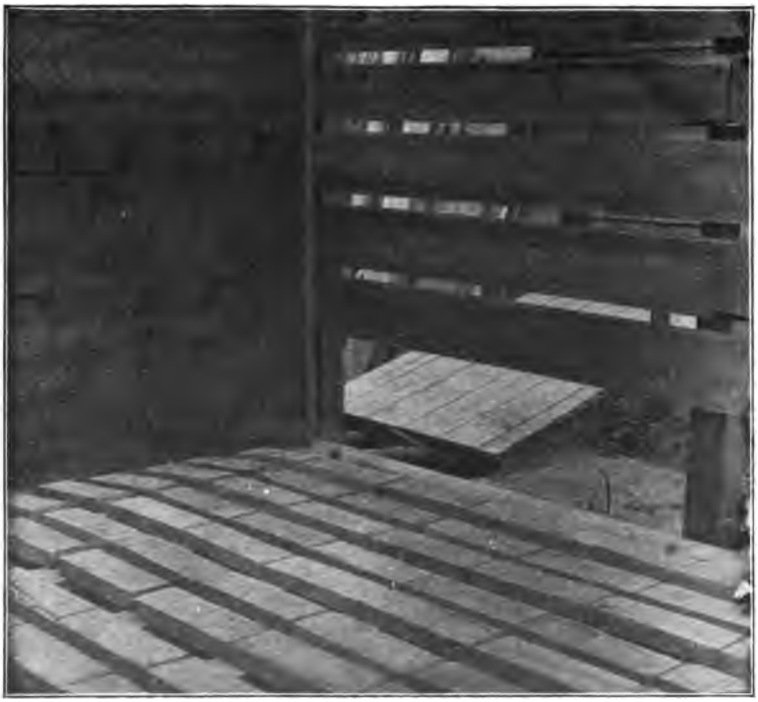


FIG. 4.—UNDERDRAINAGE SYSTEM OF TRICKLING BEDS

courses, as shown in Figure 4, to ensure good underdrainage. On these were 18 inches of bowlders and 65 inches of the main filling material — 1½ to 2-inch crushed stone.

For spraying sewage over the surface of the filter a new method was devised, which we have called the system of gravity distribution. This was described by us a year ago (Winslow and Phelps, 1906 *b*), and has since been adopted by the Massachusetts State Board of Health for its experimental filter at Andover (Clark, 1907). In this system a fine spray is produced, not by throwing the sewage up into the air from a fixed nozzle, but by discharging it downward from an opening in the bottom of a trough on to a concave metal plate, from which it splashes upward. The plan first adopted allowed four such splashing disks for each of our two filters. During this period the arrangement of the system was as follows:

The sewage from the two dosing weirs, in the distributing tank and in the septic tank No. 5, flowed out from the tank house through open wooden troughs, 3 inches in width and 3 inches deep. From these the sewage dropped into similar troughs, which ran from corner to corner of each of the two filters, joining at the centre in the form of a cross. The sewage passed into this cross at its centre, flowed out into the four arms, and dropped through ¾-inch holes to the disk distributors, four to each bed, 100 square feet in area. In each hole was a 3-inch piece of ¾-inch brass tubing, threaded on its upper inch and screwed to a lock nut inside the trough. Any inequality in distribution due to change in the level of the distributing troughs was easily corrected by changing the position of this short tube. Below the troughs the stream of sewage was protected from the wind by a box extending down for 12 inches, and at 15 inches below the trough the stream struck a shallow concave disk, from which it splashed outward. On the filter, as operated during the first year, these disks were 3 inches in diameter, and their concavity had a 6-inch radius. Each was held in place by four ¼-inch brass rods. The general arrangement of this distribution system is shown in Figure 3. It worked with fair success, breaking up the sewage well, and not requiring an excessive amount of attention. Some difficulty was experienced during the winter of 1905–06, but the beds remained in operation. On the coldest days an umbrella of ice surrounded each disk, somewhat limiting the surface distribution; but within this shield the spray worked well.

With disks spaced in this manner, however, there were considerable intermediate areas of the filter which did not receive sewage, while the portions directly under the disks were operating at excessive rates. During the summer of 1906 pooling occurred on the raw sewage side, and in November of that year we substituted for the four splashing disks

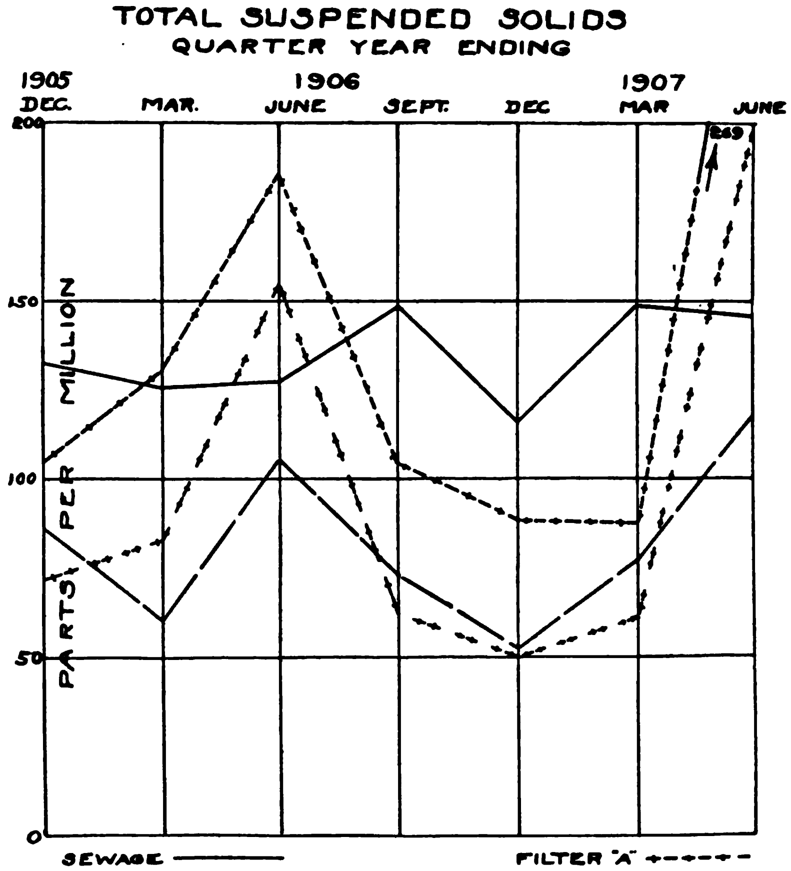


FIG. 5.—QUARTERLY VARIATIONS IN TOTAL SUSPENDED SOLIDS; SEWAGE AND EFFLUENTS

on filter A a single splashing disk of the same type, placed at the centre of the square. This single disk, with four times the discharge upon it, gave a much better distribution over the 10-foot square than its four predecessors. A detailed study of the efficiency of various trickling filter distributors (Winslow, Phelps, Story, and McRae, 1907) has shown that this type of splashing disk gives more even distribution than any other device, except certain complicated pressure nozzles with small openings. For purposes of comparison we installed on filter B a pressure nozzle of the type used in the early experiments at Columbus, Ohio. This has "a single orifice, $\frac{9}{16}$ inch in diameter, with rounded edges, above which, held by two thin arms, is an inverted

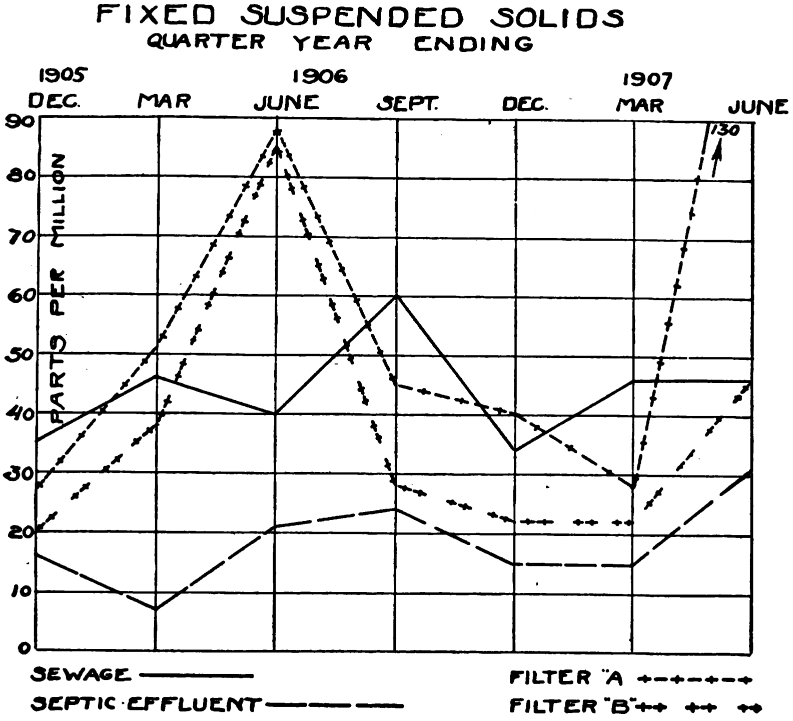


Fig. 6.—Quarterly Variations in Fixed Suspended Solids; Sewage and Effluents

90° cone, the axis of the cone coinciding with the axis of the orifice. The jet on leaving the orifice impinges against the cone and is transformed into a thin sheet, spreading out radially and breaking into a shower of fine drops" (Gregory, 1906).

A nozzle with an orifice as large as that of the Columbus sprinkler, when operated with a 4-foot head, as in our experiments, produces an excessive discharge. We therefore placed a dosing chamber between septic tank 5 and filter B, equipped with an automatic, air-locked siphon, discharging at a head of 4 feet and cutting off at 3 feet. Ball floats operated the air valves, which started and cut off the siphon sharply at the beginning and end of its discharge. In operation, the siphon tank filled in one minute twenty-six seconds, and took one minute ten seconds to empty. It discharged through a 2-inch pipe, which passed into filter B about 6 inches below its surface, rising again at the centre of the bed just above the level of the stones.

The effluent from each filter, as pointed out above, was conducted by the sloping cement floor to the inner corner of each bed, thence flowing through 1½-inch iron pipes to the lower floor of the tank house. Since January, 1906, the effluents from the filters have been sedimented in tanks of special design (see Figs. 2 and 19). Each was a simple inverted cone, with a diameter at the top of 7 feet 2 inches and a height of 4 feet. This form was not so favorable for sedimentation as the Dortmund tank, with sides parallel in the upper part, but was adopted to suit the head room and area at our disposal. At the centre of each tank a vertical tube, 6 inches in diameter, extended to within 1 foot 3½ inches of the bottom. Into the top of this tube the filter effluent was discharged, its velocity being checked by a 3-inch conical cup. The liquid passed out into the tank near the bottom of the tube through openings in its sides. Sludge was removed once a day or once in two days from the bottom of the tank by the valve shown in the diagram. The supernatant liquor rose and overflowed through two 2 × 3 inch collecting troughs. The capacity of each tank was such as to give a two hours' storage period when the filter was running at a rate of 2,000,000 gallons per acre per day.

III. METHODS OF SAMPLING AND ANALYSIS

All chemical examinations were of composite samples. Portions were collected every three hours, day and night, and at once chloroformed; these were mixed and analyzed at the end of each week. One composite sample of the sewage and of each effluent was analyzed each week for the whole period of investigation.

Turbidity and sediment were observed in each sample by the use of the Jackson turbidimeter recalibrated for coarse material (Phelps, 1905). Total and fixed solids were determined by the Gooch crucible method of Kimberly and Hommon (1906).

Total organic nitrogen values were obtained by a modified Kjeldahl method, previously described (Phelps, 1905), and during the last year nesslerizations have been made directly by a method devised by Whipple at this laboratory (Whipple, L., 1907).

Free ammonia was read by direct nesslerization against permanent standards. For nitrates the brucine method was used (Farnsteiner,

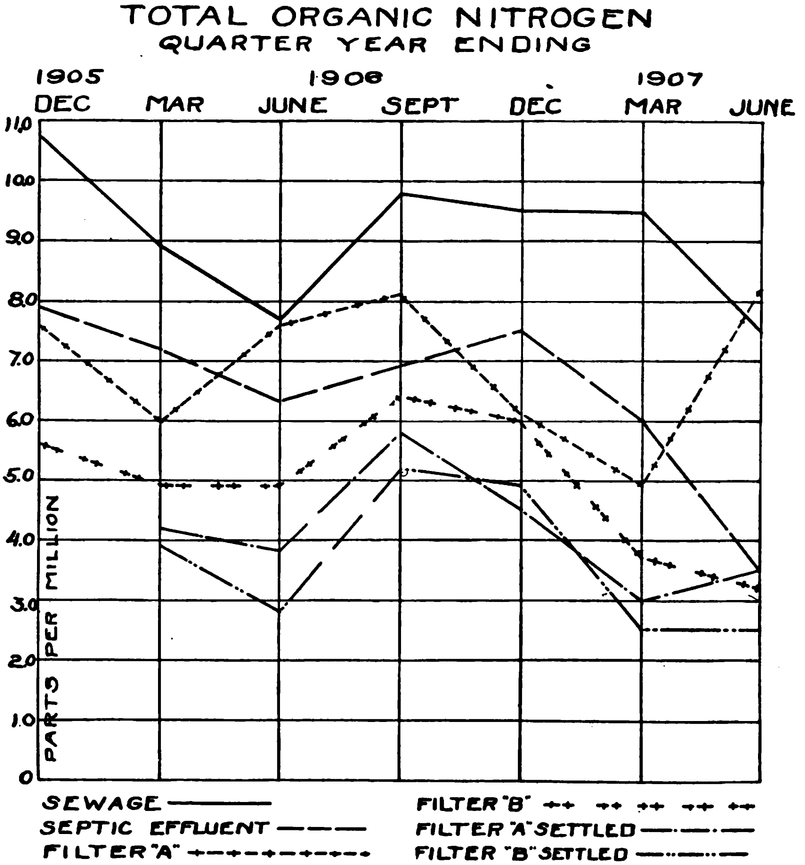


Fig. 7.—Quarterly Variations in Total Organic Nitrogen; Sewage and Effluents

1902). "Oxygen consumed" was determined after thirty minutes' heating on the water bath. This determination, and all others not specifically mentioned, were made according to the standard procedure recommended by the Committee of the American Public Health Association (1905).

Experience with the newer processes of sewage treatment has made it clear that the ordinary methods of analysis are insufficient by themselves to furnish all the information necessary with regard to the character of a sewage effluent. Our knowledge of sewage cnemistry has not yet made it possible to discriminate clearly between putrescible organic matter and those more stable "humus-like" compounds which

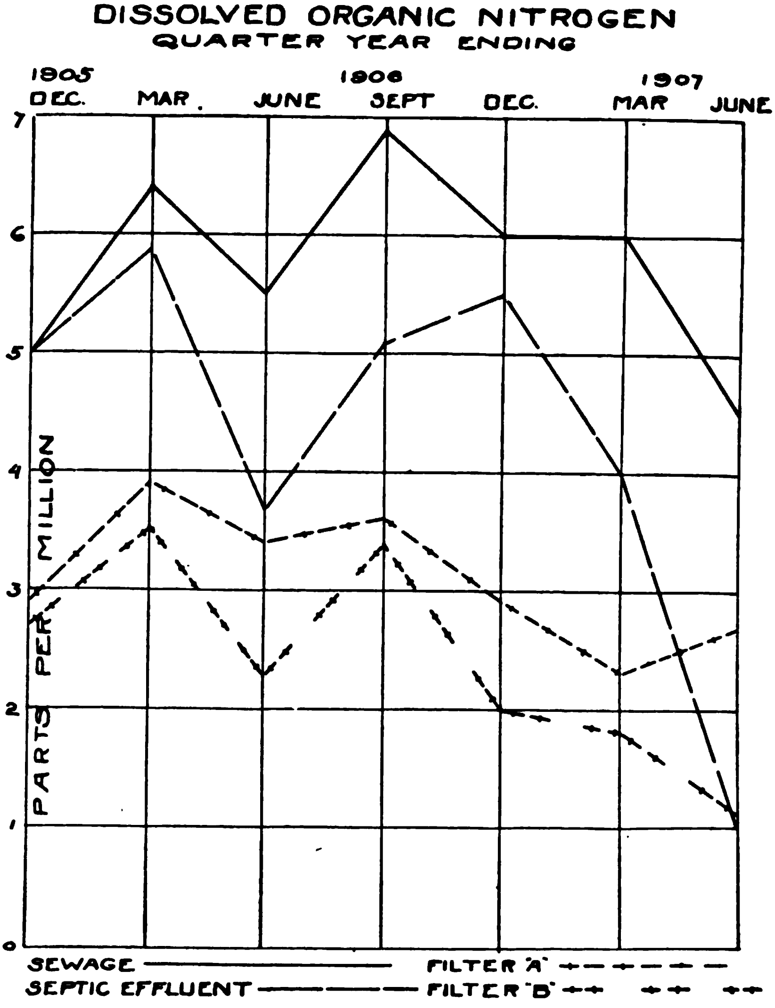


FIG. 8.—QUARTERLY VARIATIONS IN ORGANIC NITROGEN IN SOLUTION; SEWAGE AND EFFLUENTS

do not undergo putrefactive decomposition. With intermittent sand filters, effluents are obtained which contain only a small amount of organic matter of any sort; and the purity of such effluents is easily demonstrable by oxygen consumed and nitrogen determinations. With effluents from contact and trickling filters, on the other hand, the ordinary analytical data do not offer a complete criterion of quality. An effluent may contain a considerable amount of organic nitrogen and give a high value for oxygen consumed, and may yet prove of excellent quality because its organic matter is in a stable form. In such cases it becomes necessary to supplement sanitary analysis by some practical test of keeping quality. Such a test, if its technique is easy, can be made to advantage at more frequent intervals than the more elaborate chemical examinations; and in small plants, where the latter are out of the question, it alone may prove of considerable value.

Although the terms putrescible and non-putrescible stand for quite definite characteristics, yet the separation of effluents of various degrees of purity into these two classes depends to a considerable extent upon the test employed for the purpose, and none of the older tests for stability has proved wholly satisfactory. The "smell test" is inexact, and at the other extreme the determination of oxygen consumed, dissolved oxygen, nitrates, and nitrites is too cumbrous a process for routine work. Furthermore, it does not accurately measure the relation of oxidizable matter to available oxygen, since the oxygen consumed by permanganate bears a variable relation to the organic matter which is oxidizable under natural conditions. The "Manchester test" is somewhat more satisfactory; but this, too, yields abnormal results at times, and only divides effluents into two rough classes, without distinction as to their relative grade.

A new putrescibility test, simple in technique and measuring accurately and delicately the relation between available oxygen and oxidizable matter, has, therefore, been a desideratum. Such a method appears to be at hand in the methylene blue test, first devised by Spitta (1903) for the study of stream pollution, and later more thoroughly worked up by Spitta and Weldert (1906) as a test for sewage effluents. The technique employed is extremely simple. A small portion of an aqueous solution of the dye (in our experiments 1 c.c. of a .1 per cent. solution) is added to the effluent in a glass-stoppered bottle (250 c.c. capacity in our work), and the sample is then incubated either at

20° C. or at 37° C. The blue color of the solution remains practically unchanged during the period of observation until the available oxygen contained is used up and putrefactive conditions arise. At this point the dye is reduced and decolorized. The time required for decolorization is a quantitative measure of the degree of putrescibility of the sample, and the retention of the color for a period of four days or more at 20° C., or of two days at 37° C., may be taken as an indication of good stability. The criticism of Johnson, Copeland, and Kimberly (1906), that "substances other than putrescible organic matters, such as sulphide of iron and hydrogen sulphide," discharge the color of methylene blue before the putrescible matters themselves are able to act, does not

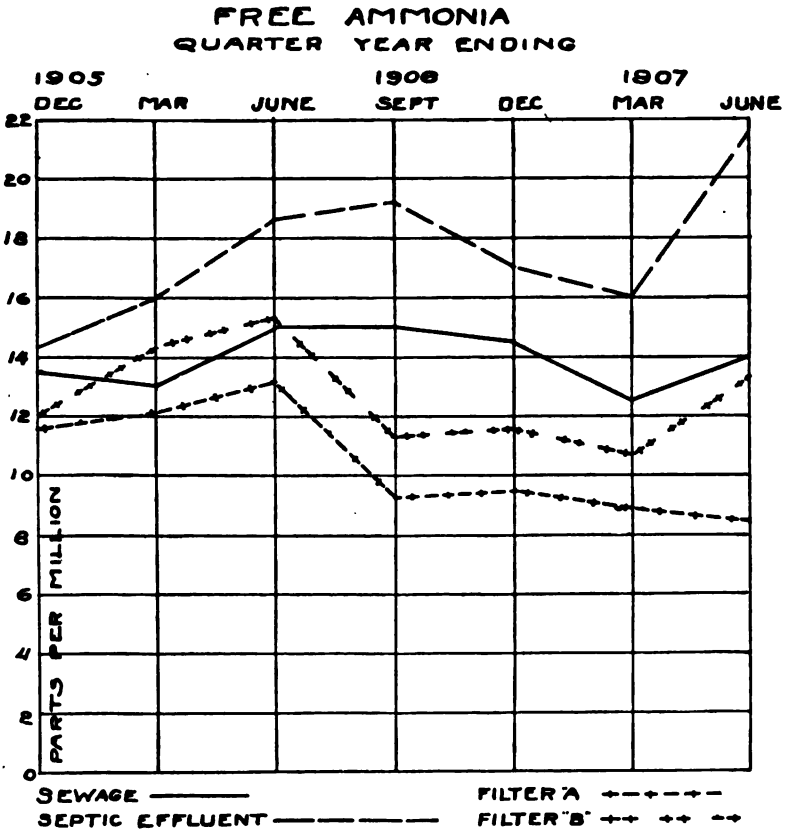


FIG. 9.—QUARTERLY VARIATIONS IN FREE AMMONIA; SEWAGE AND EFFLUENTS

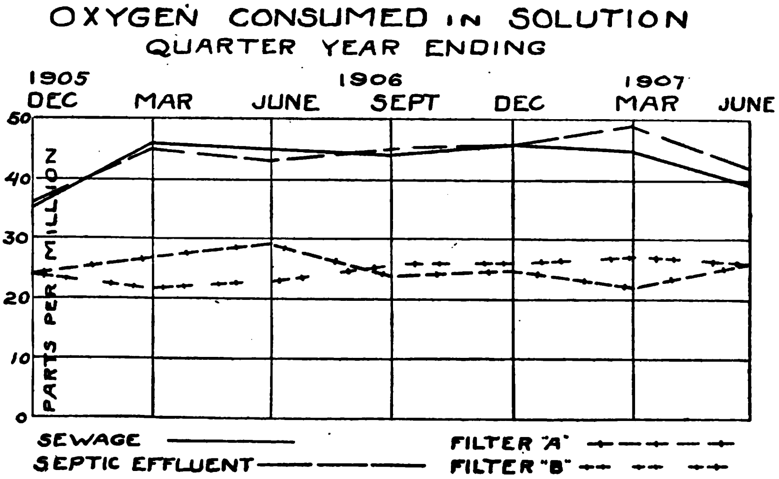


Fig. 10.—Quarterly Variations in "Oxygen Consumed" in Solution; Sewage and Effluents

seem to us well taken, since such substances are present only when the effluent as a whole is in unstable condition.

Spitta and Weldert state that putrefaction occurs from two to four times as fast at 37° as at the ordinary room temperature. We have made a somewhat fuller study of this temperature factor and of some other points in connection with the methylene blue test, which have been reported elsewhere (Phelps and Winslow, 1907). For twenty samples of trickling filter effluents, the decolorization times at 20° and at 37° were determined to the nearest half day, and the ratios of the 20° times to the 37° times are tabulated in Table I. The actual number of days varied from half a day to thirteen days at 37° C.

TABLE I—Ratio of Time Required for Decolorization of Trickling Filter Effluents at 20° to that Required at 37° C.

2.5	2.0	1.6	1.6
1.5	2.6	1.0	0.5
2.7	2.0	4.0	2.2
1.8	2.0	2.0	2.3
1.4	1.9	2.4	2.0

With the exception of two very low ratios (.5 and 1) and one high ratio (4), all these figures fall between 1.4 and 2.7, and the average of all is exactly 2. It may be considered, therefore, that methylene blue samples will generally decolorize at 37° in half the time required at 20°.

It must be remembered in interpreting this table, and all others in which comparative decolorization tests are recorded, that each test must be made in a separate bottle; and since the sampling of sewage is never perfect, there will be variations due to chance differences in the several bottles compared. This is particularly true in work like that later cited in Table III, in which decolorization tests were made in one bottle and analytical data determined on another.

In general, it appears from Table I that results obtained at 20° bear to those obtained at 37° the ratio of 2 to 1. Stable samples are stable at any temperature. In grading putrescible samples, however, it is evidently necessary to select one temperature and to use it for all comparative work. The temperature of the body has the advantage that it gives results more promptly. Any sample which does not decolorize in four days is certainly stable, and a two days' limit would include almost all putrescible effluents. The 20° method, however, from the very fact that it is slower, permits a more delicate measurement of the relative differences between putrescible effluents of different grades. Decolorization here may occur from the first up to the fourteenth day. Slight variations from 20° will not introduce a serious error in the test, and diffuse daylight does not appreciably interfere with it. A series of control experiments made in daylight, in a box at the room temperature (in summer), and in an incubator at 20°, are recorded in Table II, and indicate that the test could be made in an ordinary room with fairly controlled temperature. Experiments made during cold weather in a small frame building heated by a stove showed marked abnormalities, on the other hand, and emphasize the fact that temperature variations cannot be allowed to go too far.

TABLE II — Effect of Temperature and Light on Methylene Blue Test

Average Time of Decolorization in Days

Week ending	Effluent A.			Effluent B.			Effluent C.		
	Summer temperature.		20° dark.	Summer temperature.		20° dark.	Summer temperature.		20° dark.
	Daylight.	Dark.		Daylight.	Dark.		Daylight.	Dark.	
June 27	1	2	2	11	11	7	2	3	3
July 2	3	3	3	12	12	10	3	3	4
July 9	2	2	2	2	3	2	2	3	3
July 16	2	3	2	9	12	10	13	13	13
July 23	2	3	4	7	8	7	14+	14+	14+
July 30	2	5	6	13	14+	13	12	14+	13
August 6	2	3	3	6	6	6	14+	13	13
August 13	3	5	4	13	13	12	13	13	13
August 20	10	5	5	14+	14+	14+	14+	14+	14+

With regard to the significance of the methylene blue test in relation to the specific chemical changes which go on in putrefaction, Spitta and Weldert have little to say, beyond pointing out that decolorization takes place only after all the available oxygen has been exhausted, which point they illustrate by a single experiment. We have studied the relation between decolorization and chemical processes a little more fully, and the results of the examination of five samples of trickling effluents are shown in Table III.

TABLE III—OXYGEN CHANGES IN PUTRESCIBLE TRICKLING FILTER EFFLUENTS

Decolorization of Methylene Blue Indicated by *

(Parts per Million)

Sample.	Days at 37°.	Dissolved oxygen.	Nitrogen as		Oxygen consumed fifteen minutes cold.
			Nitrates.	Nitrites.	
1	Initial	8.0	3.0	1.0	5.0
	1*	0.0	0.0	0.3	5.0
	2	. .	. .	0.1	. .
2	Initial	10.0	3.0	1.0	4.0
	1	1.0	3.0	1.0	4.0
	2	0.0	0.0	1.0	3.4
	3	0.0	. .	0.1	4.0
	4	. .	. .	0.06	4.0
	5	. .	. .	0.03	3.8
	7	. .	. .	0.0	4.0
	8	. .	. .	. .	4.8
	9	. .	. .	. .	6.3
3	Initial	8.0	3.5	0.5	6.0
	1	1.0	3.0	1.2	4.8
	2	1.2	3 0	1.4	5.7
	3	0.4	3.0	1.0	5.2
	4	0.0	0.0	0.0	5.0
	7*	. .	. .	. .	. .
4. M. B. sample lost	Initial	10.8	4.0	0.3	4.2
	1	1.8	3.0	0.3	4.5
	2	0.2	3.0	0.5	4.4
	3	0.2	3.0	0.1	3.4
	4	0.6	2.5	0.1	2.5
	5	0.0	1.0	0.1	4.3
	6	. .	0.0	0.05	3.6
	7	. .	0.0	0.0	2.7
5	Initial	9.2	3.0	0.6	5.2
	1	1.2	2.5	0.0	5.3
	2*	0.0	0.0	0.0	5.3
	3	0.0	0.0	0.0	5.7
	4	. .	. .	. .	4.6
	5	. .	. .	. .	3.9
	6	. .	. .	. .	6.1
	7	. .	. .	. .	6.9

It is evident that the free dissolved oxygen is absorbed first by the organic compounds present; then the nitrates disappear, and finally the nitrites; and the methylene blue is generally attacked just after the disappearance of the latter. Sometimes, as in samples 2 and 3, there is a considerable delay between the disappearance of nitrites and the decolorization of the methylene blue. This may probably occur when the organic matter and the available oxygen are about evenly balanced, so that after the exhaustion of the oxygen putrefactive changes are set up somewhat slowly. Whenever the methylene blue is decolorized it may be assumed that all oxygen as nitrates or nitrites has disappeared; and, *vice versa*, when the blue color is retained traces of these bodies are generally present. This condition is illustrated in Table IV for trickling filter effluents which did not decolorize after fourteen days at 37°. It will be noticed that these effluents contained a considerable amount of organic matter as measured by "oxygen consumed," and that in all cases there was a marked reduction in oxygen content during incubation. Nitrites and nitrates remained, however, in all but three cases. This condition may be compared, for contrast, with that of the putrescible effluents in Table III and with four other sets of analyses quoted in Table V.

TABLE IV—OXYGEN AND OXIDIZABLE MATTER IN STABLE TRICKLING FILTER EFFLUENTS

(Parts per Million)

INITIAL COMPOSITION.				AFTER FOURTEEN DAYS AT 37°.			
Dissolved oxygen.	Nitrogen as		Oxygen consumed fifteen minutes cold.	Dissolved oxygen.	Nitrogen as		Oxygen consumed fifteen minutes cold.
	Nitrites.	Nitrates.			Nitrites.	Nitrates.	
6.8	4.0	5.5	5.6	0.0	0.0	0.0	3.2
2.8	3.0	5.5	8.4	0.3	3.5	1.5	7.6
5.2*	0.0	15.0	4.4	4.0	0.1	15.0	4.4
7.2	5.0	3.0	18.8	0.1	0.4	1.5	8.9
4.4	3.0	3.0	5.2	0.0	0.1	1.5	3.5
7.2	4.0	6.0	16.8	0.1	0.5	5.0	15.8
4.6	3.0	6.0	10.4	0.0	1.5	1.0	12.2
4.4	2.5	5.5	9.6	0.0	0.0	0.0	7.0
4.4	3.0	5.5	8.4	0.2	0.8	1.5	2.0
6.6	1.0	2.0	17.4	0.0	3.0	0.1	2.6
4.2	1.5	3.0	7.2	0.0	1.0	0.0	0.6
5.7	2.5	3.0	8.4	0.0	0.0	0.0	5.2
4.3	3.0	4.0	7.0	0.6	6.0	2.0	7.7

* Sand filter effluent.

TABLE V—OXYGEN AND OXIDIZABLE ORGANIC MATTER IN PUTRESCIBLE TRICKLING FILTER EFFLUENTS

(Parts per Million)

INITIAL COMPOSITION				Age, hours at 37°.	AFTER DECOLORIZATION.			
Dissolved oxygen.	Nitrogen as		Oxygen consumed fifteen minutes cold.		Dissolved oxygen.	Nitrogen as		Oxygen consumed fifteen minutes cold.
	Nitrites.	Nitrates.				Nitrites.	Nitrates.	
4.3	2.0	3.0	24.4	24	0	0	1.5	25.2
1.1	2.5	3.0	14.0	12	0	0	0.0	14.0
5.8	1.5	5.0	15.6	40	0	0	0.0	16.0
3.8	1.5	5.0	8.8	120	0	0	0.0	2.8

An interesting practical question arises as to the relation of the methylene blue test to the original constitution of the effluent as determined by chemical analysis. If our methods were sufficiently precise we should be able to predict from determinations of organic matter and oxygen just what the result of their interaction will be. The condition of an effluent, as indicated by the relation of oxygen consumed to available oxygen, corresponds, however, only roughly to its putrescibility. Comparing Tables III, IV, and V, a general difference between the stable and putrescible samples is apparent, the former, in Table IV, having on the whole a considerably higher ratio of available oxygen to oxygen consumed than the latter, as shown in Tables III and V. There are, however, many samples which contradict this general rule, and we have been unable on any basis of calculation to find a marked and constant difference in analysis between stable and putrescible samples. This has been still more forcibly brought out in our routine experiments. For a considerable period, as will be shown later on, one of our trickling beds gave stable effluents, while those from the other bed were putrefactive. Yet at the same time there were no

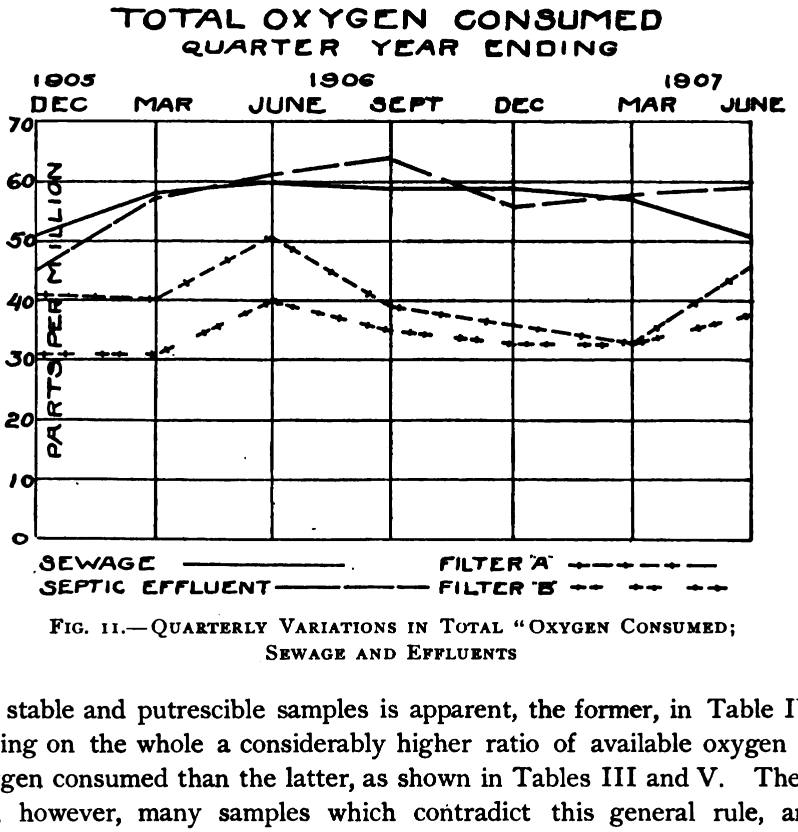


FIG. 11.—QUARTERLY VARIATIONS IN TOTAL "OXYGEN CONSUMED; SEWAGE AND EFFLUENTS

significant differences in the nitrogen and oxygen-consumed values of the two effluents. It is evident that certain factors affect the stability of an effluent which are not revealed by the ordinary methods of sanitary chemistry; and since this property of stability is precisely the point of greatest significance in sewage purification, the methylene blue test seems to us of supreme value in judging the work of trickling filters. We have therefore used it since February, 1906, in all our routine work, making daily tests of crude sewage, septic effluent, and trickling effluents before and after sedimentation.

Bacterial samples of sewage and effluents were taken three times a week from October, 1906, to April, 1907, and counts were made on litmus lactose gelatin at 20°. As a presumptive test for *B. coli* the bile medium of Jackson (1906) was used. As in Jackson's work, natural ox-bile was used, with the addition of 1 per cent. lactose; but we incubated our tubes for three days instead of two, which time he first recommended. The reason for this was that a series of sixty preliminary tests gave the results shown in Table VI, which indicated that forty-eight hours is too short a time to allow the development of all the gas-forming organisms present.

TABLE VI — POSITIVE REACTIONS WITH BILE MEDIUM AT VARIOUS PERIODS

Period of incubation.	Average positive results.	Average per cent. gas.
48 hours	77 per cent.	10
72 hours	92 per cent.	24
96 hours	95 per cent.	38

For the first three months of regular use, October to December, 1906, the bile test gave good results, yielding positive tests in crude sewage in one-millionth of a cubic centimeter. In January, however, either the number of colon bacilli greatly decreased or the bile obtained was antiseptic. In April *B. coli* could not be isolated by this method from one one-thousandth of a centimeter of sewage.

IV. COMPOSITION OF CRUDE SEWAGE

The sewage of the south Metropolitan district was made slightly weaker in October, 1904, by the diversion of the flow from its more

outlying areas into the new Peddock's Island high level sewer. Comparing the analyses of 1904–05 with those of 1903–04, a diminution of about 10 per cent. in the strength of the sewage was apparent. The analyses made since 1905 are not comparable with preceding ones, since both nitrogen and oxygen-consumed methods were changed at the beginning of the present investigation.

TABLE VII—COMPOSITION OF CRUDE SEWAGE

Quarterly Averages

(Parts per Million)

Year.	Quarter.	Temperature F.	Turbidity.	Sediment.	Suspended Solids. Total.	Suspended Solids. Fixed.	Organic Nitrogen. Total.	Organic Nitrogen. Solution.	Free ammonia.	Nitrites.	Nitrates.	Oxygen Consumed. Total.	Oxygen Consumed. Solution.	Oxygen dissolved.
1905	4	52	235	120	133	35	10.7	5.0	13.5	0.1	0.2	51	35	3.2
1906	1	43	245	110	125	46	8.9	6.4	13.1	0.0	0.4	58	46	5.6
1906	2	56	285	140	127	40	7.7	5.5	15.0	0.0	0.3	60	45	2.0
1906	3	72	314	160	149	60	9.8	6.9	15.0	0.1	0.0	59	44	0.2
1906	4	54	295	125	116	34	9.5	6.0	14.5	0.0	0.1	59	46	2.5
1907	1	42	320	115	149	46	9.5	6.0	12.5	0.1	0.2	57	45	6.5
1907	2	55	260	80	146	46	7.5	4.5	14.0	0.0	0.0	51	39	3.8
Average	. .	53	279	121	135	44	9.1	5.8	13.9	0.0	0.2	56	43	3.4

The average analysis of the crude sewage for each quarterly period from October, 1905, to June, 1907, is shown in Table VII. Most of the constituents were fairly constant throughout the whole period. The first quarter of the year is characterized by a marked increase of dissolved oxygen and a diminution of free ammonia, due to the inhibition of bacterial decompositions by low temperature. The second quarter of the year is marked by a decrease in organic nitrogen, due to the greater dilution of house sewage at that season with street washings and melted snow. At the same time, the free ammonia value rises with the rising temperature and more active bacterial decomposition. On the whole, the analyses indicate that the sewage used in these experiments was of about average strength for an American sewage. It is weaker than the sewages of many small Massachusetts cities where the proportion of house sewage is large. On the other hand, its organic content is slightly greater than that of the sewage of Columbus (Johnson, 1905). The "oxygen-consumed" value is 56 for Boston against 51 for Columbus; the free ammonia value 13.9 against 11; and the Kjeldahl nitrogen figure 9.1 against 9. Columbus sewage contains only 79 parts of volatile suspended solids, while Boston sewage has 91, but shows 130 parts of fixed suspended solids against 44 parts for Boston.

The average bacterial content of the crude sewage from October, 1906, to April, 1907, inclusive, was 1,200,000 per cubic centimeter as determined by plating on litmus-lactose-gelatin at 20°. Of these, 20 per cent. were acid formers and 8 per cent. liquefiers. The average number of bacteria during the summer months would no doubt have been considerably higher, as shown by previous investigations (Winslow, 1905).

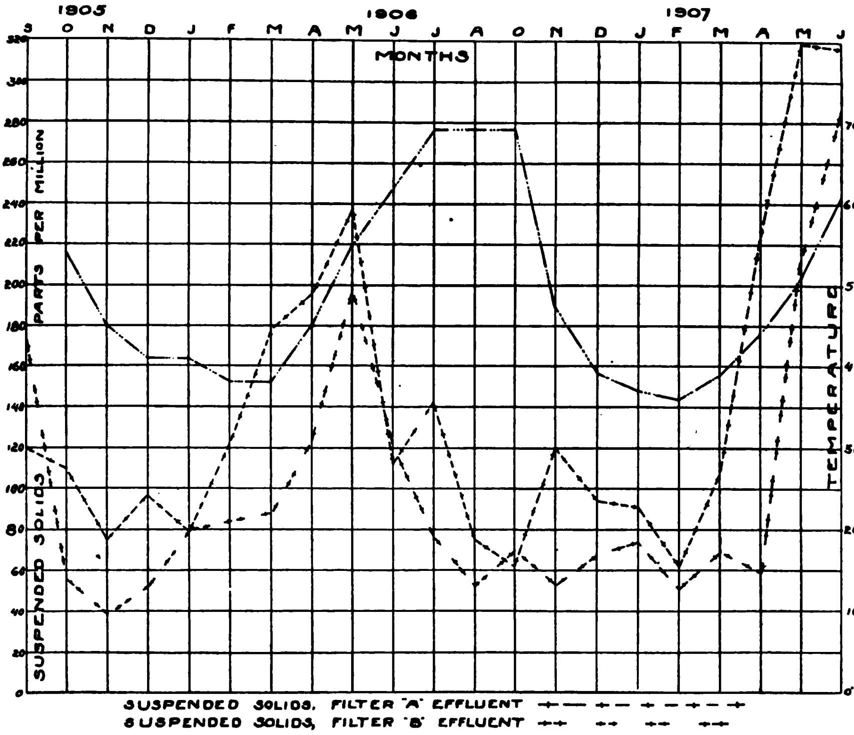


FIG. 12.—MONTHLY VARIATIONS IN TEMPERATURE AND SUSPENDED SOLIDS IN TRICKLING EFFLUENTS

V. RESULTS OF SEPTIC TREATMENT

The septic system used in this investigation consisted, as already stated, of five tanks, each 6 feet long, 4 feet broad, and 3 feet deep.

These were connected by 2.5-inch pipes, and each contained a large baffle projecting from one corner nearly to the opposite corner, as shown in Figure 1. The path of a particle of liquid in each tank was about 10 feet long, and the action of all five tanks approximated that of a single tank 50 feet long and 3 feet wide.

Experiments were begun August 29, 1905, with all five tanks in series, but Nos. 2 and 3 were cut out October 19, 1905, and for one year three tanks alone were used. October 15, 1906, the two extra tanks were again placed in connection with the others. For the first year the total septic period was about seven hours, and for the last eight months it was twelve hours.

On the second day after the tanks were first started a scum formed, which gradually thickened so that after seven days it was sufficiently strong to retain gas bubbles. Masses of black sludge were observed rising to the top of the tank and floating. Conditions remained almost the same during the entire course of the investigation. The scum varied somewhat in amount from time to time, never attaining a thickness of over a quarter of an inch, and often entirely disappearing. Tanks Nos. 1, 4, and 5 operated for twenty months, and Tanks Nos. 2 and 3 for eight months, without any superficial indication of undue accumulation of solids.

There are two distinct processes involved in septic treatment — the removal of suspended solids by sedimentation and the liquefaction of the organic matter by bacterial action. The first of these processes is measured by the character of the effluent discharged, and Table VIII indicates what our septic tanks accomplished from this standpoint.

TABLE VIII — COMPOSITION OF SEPTIC EFFLUENT

Quarterly Averages

(Parts per Million)

Year.	Quarter.	Temperature F.	Turbidity.	Sediment.	SUSPENDED SOLIDS.		ORGANIC NITROGEN.		Free ammonia.	Nitrites.	Nitrates.	OXYGEN CONSUMED.	
					Total.	Fixed.	Total.	Solution.				Total.	Solution.
1905	4	52	175	70	86	16	7.9	5.0	14.3	. .	. .	45	36
1906	1	43	190	80	60	7	7.2	5.9	16.0	. .	. .	57	45
1906	2	58	215	93	105	21	6.3	3.7	18.6	0.1	0.1	61	43
1906	3	73	203	98	72	24	6.9	5.1	19.2	0.0	0.0	64	45
1906	4	53	220	85	52	15	7.5	5.5	17.0	0.0	0.0	56	46
1907	1	44	240	80	77	15	6.0	4.0	16.0	0.0	0.0	58	49
1907	2	55	250	105	118	31	3.5	1.0	21.5	0.0	0.0	59	42
Average	. .	54	213	87	81	18	6.5	4.3	17.5	0.0	0.0	57	44

Comparing the analyses of the septic effluent with those of the crude sewage (Table VII), it appears that the tank system removed 40 per cent. of the total suspended solids and 59 per cent. of the fixed suspended solids. There was a 28 per cent. decrease in the total organic nitrogen and, curiously enough, a 26 per cent. decrease in the organic nitrogen in solution. This latter point shows that the septic tank exercises a certain bacterial purification besides its primary sedimentary action. On the other hand, and, indeed, as a result of the decomposition of organic nitrogen, the tank effluent showed a 26 per cent. increase in free ammonia. "Oxygen-consumed" values were about the same in the effluent as in the crude sewage.

Some interesting points are brought out by a study of the quarterly analyses, which are plotted, for the more important constituents in Figures 5 to 11. Taking the organic matter first, it appears from Figure 7 that the removal of this type of material as a whole does not vary materially from season to season. Figure 8 shows, however, that the soluble organic nitrogen is much more strikingly reduced during the second and third quarters of the year than at other times. At the same season, as appears from Figure 9, the production of free ammonia in the septic tank increases rapidly. These phenomena are both, of course, correlated with the increased bacterial action which follows the setting in of warm weather. The same close relation between septic action and temperature has been shown by Kinnicutt and Eddy (1902), in their study of the gases formed in a closed septic tank. This point is of much importance in plans for the use of septic tanks under varying climatic conditions, as Fowler has pointed out. In India septic tanks work to perfection, while in Russia they are unsatisfactory (Fowler, 1907).

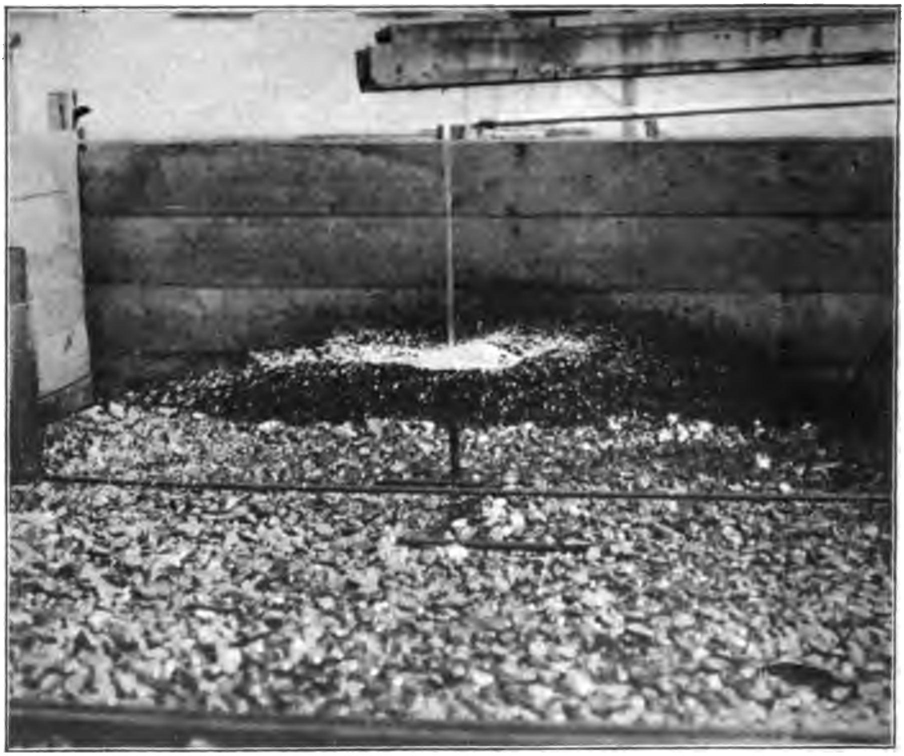


FIG. 13.—VIEW OF GRAVITY DISTRIBUTOR IN OPERATION

Turning to Figures 5 and 6, it appears that the septic tank is least efficient in the removal of suspended solids just at the time when its period of greatest biological activity begins; that is, in the second quarter of the year. This we explain by the active fermentation which manifests itself in the lifting of masses of sludge from the bottom of the tank and the general stirring up of its contents, and naturally leads to the discharge in the effluent of a larger proportion of suspended matter than at times of comparative rest.

Figures 10 and 11, showing the relations for carbonaceous matter, as indicated by the "oxygen-consumed" test, are somewhat surprising from the close parallelism which they indicate between the sewage and the septic effluent. Matter of this class was apparently not appreciably decomposed and scarcely affected by sedimentation. The alternative to this conclusion is to suppose an actual removal of carbonaceous matter, balanced by the production of new reducing bodies. It is difficult to suppose that such a process would yield the uniform results indicated by Figure 11. English septic tanks show, as a rule, a distinct loss of "oxygen consumed" (Winslow and Phelps, 1906*a*).

We believe that differences in results of this sort at different places are to be explained in part, at least, by the different methods in use in determining the oxygen-consumed value. The more vigorous the oxidizing action of the permanganate the more nearly the results approach the true total carbon values. Changes in the nature of the carbonaceous matter, which do not affect its total amount, are quite obscured by analytical procedures like the present standard method, involving a long period (half an hour) of boiling. The less intense English cold methods, on the other hand, give results which are markedly dependent on the nature as well as the amount of the carbonaceous matter. They are therefore extremely valuable in studying those slight chemical changes which accompany sewage purification. It is even possible to detect a difference between the results of the present standard method, involving a half-hour boiling, and the older Massachusetts method of two minutes' boiling, used in this laboratory two years ago. By the latter method our six septic tanks showed an average removal during two years of 14 per cent. of the total "oxygen consumed," as compared with no removal whatever in the present series as determined by the present standard method.

One notable point brought out by these diagrams is the fact that the change from three septic tanks to five, made in October, 1906, did not appreciably alter the character of the effluent. Judged by the analytical results, seven hours' storage is apparently sufficient to effect all the septic changes which will take place in this particular sewage.

The extent to which it accumulates sludge is quite as important a point in the working of a septic tank as the character of its effluent. This point was studied by the method used in our previous investigations. On three occasions — June 30, 1906, when the tanks had run eight months; July 1, 1907, after they had run twenty months;

and August 3, 1907, after they had stood without the addition of fresh sewage for one month — the inlets and outlets of all the tanks were closed and the contents of each tank, including scum and sludge, were thoroughly stirred. Samples of the suspension thus produced were then analyzed in order to gain an idea of the material which had accumulated during the whole period of operation. The results are shown in Table IX. The analyses refer to the total liquid and solid contents of each tank at the time its operation ceased, a suspension containing about 98 per cent. of water. The sludge was estimated by allowing the mixed tank contents to settle in a cylindrical vessel for twenty-four hours and observing the relation between the thick sediment produced and the clear supernatant liquid. The thickness of the sludge (which as measured by this method includes the scum) was about 4 inches after the first year of operation, and averaged nearly a foot at the end of the investigation.

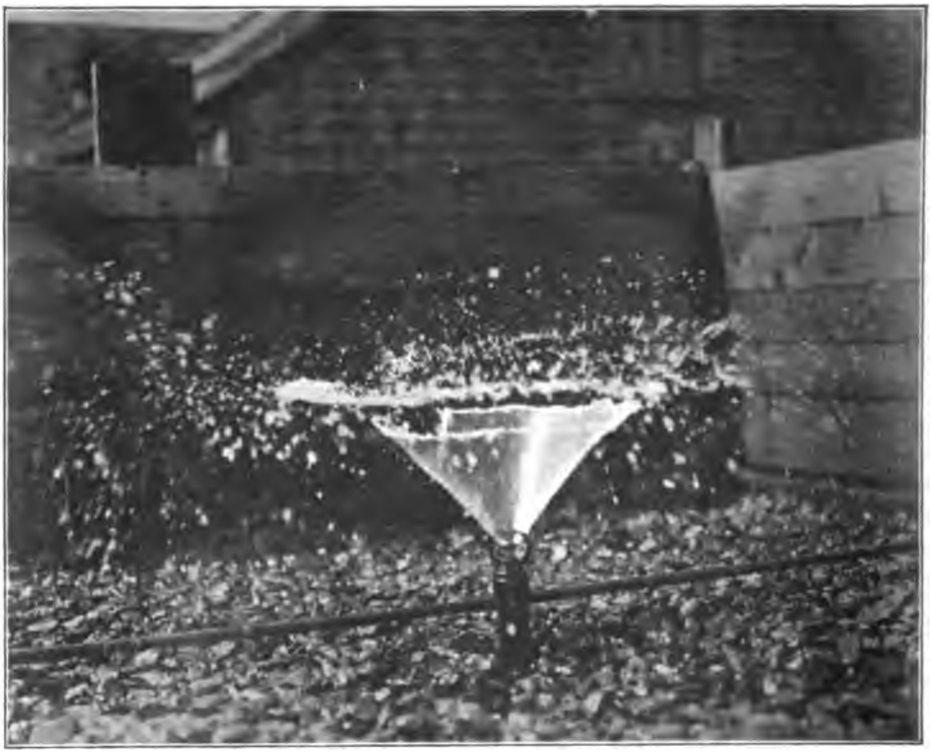


Fig. 14.— View of Columbus Sprinkler in Operation

TABLE IX — ANALYSES OF SEPTIC TANK CONTENTS, INCLUDING SLUDGE AND SCUM

(Parts per Million)

Date.	Tank.	Liquid contentr.	Depth of sludge.	SOLIDS.		Organic nitrogen.	Oxygen consumed.
				Total.	Volatile.		
		Gallons.	Feet.				
June, 1906	1	518	0.3	7,600	3,900	. .	770
	4	498	0.2	4,400	2,200	. .	520
	5	444	0.3	9,900	4,500	. .	1,200
July, 1907	1	450	0.9	. .	. .	1,000	9,000
	2	468	1.1	. .	. .	1,400	9,900
	3	468	0.8	. .	. .	860	7,300
	4	486	0.9	. .	. .	1,100	9,400
	5	495	0.5	. .	. .	610	5,200
August, 1907	1	450	0.7	25,000	4,000	380	4,900
	2	468	0.5	18,000	3,200	290	3,700
	3	468	1.1	46,000	9,700	590	7,400
	4	486	1.1	38,000	3,000	560	7,000
	5	495	1.2	44,000	7,700	540	7,000

It appears from Table IX that the differences between the individual tanks at a given time were only such as would be expected from sampling errors with a thick organic suspension. The composition of the liquid in the three (or five) tanks was approximately the same at a given time, and the system may therefore be considered as a whole.

Comparing the 1906 analyses with those made at the end of the twenty-one months, it is evident that the amount of fixed solids in the tanks increased considerably during the course of the experiments. The same thing is true of oxygen consumed, and in each case the increase is more than proportional to the time of operation. This is, perhaps, partly due to the fact that during the first year the system was operated with a seven-hour period, and during the second year with a twelve-hour period. Our previous investigations have shown that long storage periods are associated with the undue accumulation of sludge (Winslow and Phelps, 1906 *a*), and similar observations have been made at Lawrence (Massachusetts, 1901).

Over-septic sewage may inhibit the action of the liquefying organisms, or there may occur a secondary precipitation of inorganic reducing bodies, iron compounds, for example. Too much aging seems, at any rate, to be associated with an increase in fixed solids and in oxygen consumed. The same phenomenon occurs when sludge is stored in a bottle in the laboratory. Table X shows the results of successive analyses of two samples of septic sludge so tested in July, 1906. The mere aging of the septic system as a whole might tend to increase such reactions during the second year. At London and at Leeds a gradual deterioration of septic effluents was noted with the increasing age of the tanks (Winslow and Phelps, 1906*a*).

TABLE X — SEPTIC SLUDGE BEFORE AND AFTER STORAGE

(Parts per Million)

Storage, days.	Fixed solids.	Volatile solids.	Oxygen consumed.
0	33,600	35,880	7,040
2	43,500	36,400	9,960
5	52,560	35,840	10,960
0	30,900	31,400	7,360
2	34,200	26,3'0	8,340

The effect of a month's rest of the septic tanks is shown by a comparison of the analyses for July and August, 1907. The total amount of sludge did not change materially, but the oxygen-consumed value was slightly reduced and the organic nitrogen dropped more than 50 per cent. Suspended solids were still, however, very high. It is evident that after nearly two years of operation our septic system was approaching the limit of its capacity. With a foot of sludge in the bottom of the tanks, equal to 35 per cent. of the tank capacity, the effluent might soon have been expected to deteriorate. It is probable, therefore, that septic tanks operating with Boston sewage would require emptying once in two or three years.

A somewhat clearer idea of the actual work of the septic tanks may be gained by reducing the analytical data to the actual quantities they represent. During the twenty-one months of the investigation 2,490,000 gallons of sewage passed through the septic system. This volume of sewage carried 2,790 pounds of suspended solids (of which 1,880 pounds were volatile) and 188 pounds of organic nitrogen. The effluent leaving the tanks bore 1,670 pounds of suspended solids (of which 1,300 pounds were volatile) and 134 pounds of organic nitrogen. The septic system, therefore, removed 1,120 pounds of solid matter (580 pounds volatile and 540 pounds fixed) and fifty-four pounds of organic nitrogen.

TABLE XI—STORAGE AND DECOMPOSITION OF ORGANIC MATTER IN SEPTIC TANKS

Percentages of Sewage Value

	Suspended Solids.		Organic nitrogen.
	Fixed.	Volatile.	
Sewage	100	100	100
Septic effluent	41	69	71
Removed by tank	59	31	29
Stored in tank	62	6	4
Decomposed in tanks	3*	25	25

* Precipitated.

There were left stored in the tanks, after twenty-one months of operation and one month's rest, 675 pounds of solids (of which 109 pounds were volatile) and eight pounds of organic nitrogen. Four hundred and seventy-one pounds of volatile solids and forty-six pounds of organic nitrogen had, therefore, been decomposed in the tanks, while the fixed solids had been increased by twenty-six pounds—additional evidence of the precipitation of inorganic solids previously alluded to. These results are shown in the form of percentages in Table XI, where their relations may be somewhat more easily grasped.

On the whole, it appears that the septic tank will remove nearly two-thirds of the fixed suspended solids from Boston sewage and about one-third of the organic matter, as measured by suspended volatile solids or organic nitrogen. Of the organic matter in the sludge thus removed four-fifths is decomposed. The fixed solids, on the other hand, accumulate to such an extent as to require the cleaning of the tanks once in two or three years. A seven-hour period gives as good an effluent as a twelve-hour one, and probably produces a less accumulation of sludge. All these results harmonize very closely with those obtained in the previous experiments of 1903–05 (Winslow and Phelps, 1906 *a*).

The average bacterial content of the septic effluent for the eight months, October, 1906, to April, 1907, was 750,000, a reduction of

38 per cent. as compared with the crude sewage. The proportion of acid formers and liquefiers was about the same as in the sewage, 16 per cent. of the former and 8 per cent. of the latter. The results of the bile test for *B. coli* were somewhat erratic. In October and November the number of gas formers was apparently increased by passage through the septic tanks; in October 19 per cent. of the tests in one-millionth of a cubic centimeter were positive in the sewage and 56 per cent. were positive in the septic effluent; in November 9 per cent. of the tests were positive in the sewage and 38 per cent. in the septic effluent. From December to April negative results were consistently obtained in one-millionth of a cubic centimeter. The crude sewage showed 19 per cent. of positive results in December, 11 per cent. in January, 12 per cent. in February, and none in March and April.

VI. Results of Trickling Filtration

The general construction of the two trickling filters has been already described and is shown in Figures 1, 2, and 3. Both were 8 feet deep, and the surface of each was a square, approximately 10 feet on a side. Both were filled with 1½ to 2 inch broken stone. Filter A received crude sewage and filter B septic effluent.

The exact area of filter A was 98.47 square feet, or .00226 acre; of filter B, 94.57 square feet, or .00217 acre. The rate on each bed during operation was 200 gallons per hour for twenty-two hours each day, a gross rate of approximately 2,000,000 gallons per acre per day. During the summer months the plant was shut down on Sundays, while in winter it was run continuously to obviate the chilling of the bacteria in the empty bed. In September, 1906, the whole plant was shut down for two weeks. Careful records of operating time show that the filters were operated for 12,450 hours in the ninety-one weeks from October 1, 1905, to July 1, 1907. This means an average operation of 137 hours a week, or 5.7 days. The total amount of sewage applied to each bed was 2,490,000 gallons — an average of 3,900 gallons a day. The net rate of operation was, therefore, 1,730,000 gallons per acre per day for filter A, and 1,800,000 gallons per acre per day for filter B.

Each filter was at first equipped (August 21, 1905) with four gravity distributors — copper disks, 3 inches in diameter, with a 6-inch radius of curvature. Those on the septic side corroded badly, and on November 1 four new disks were substituted on each side to test the durability of various alloys. Pure aluminum proved as bad as copper, but three alloys — brass, bronze, and a nickel bronze — gave no indications of corrosion. No trouble was experienced in any case on the crude sewage filter.

The winter of 1905–06 was a fairly mild one, as may be seen by reference to Table XII, in which the maximum and minimum temperatures for the three winter months are tabulated from the published data of the United States Weather Bureau. The "normal" December temperature at Boston for different days of the month varies between 27° and 35° F., the normal January temperature between 24° and 31° F., the normal February temperature between 23° and 31° F. Ice first formed on the filters November 11, 1905, and by January 10 all the sprinklers were covered with mantles of ice — tubes about 1 foot in inside diameter, continuous from the box above the sprinkler to the surface of the bed. Inside this mantle all the sewage was distributed, and the areas between the four 1-foot circles were covered with 6 to 8 inches of ice.

In August, 1906, a slight pooling of sewage was observed on the beds, particularly on filter A. On digging into the beds it appeared that just under the four sprinklers the stones were clogged with black sludge. This layer was turned over with a pick to about a foot in depth on filter A, but no material was removed. It was evident, however, that the four sprinkling disks, each operating at a rate of .8 gallon per minute, were not distributing satisfactorily; and this experience illustrates the importance of proper distribution in trickling filters. In the middle of November changes were made in the distributing system as previously described. Filter A was equipped with a single gravity disk just like the four previously used, and filter B was equipped with a pressure nozzle of the Columbus type, and dosed intermittently from a siphon tank. The new distributors in normal operation are shown in Figures 13 and 14.

After this change no difficulty was experienced in securing a satisfactory working of the filters at all times. There was no tendency to surface pooling; analytical results were good, and, as will be seen later, there was no important accumulation of material in the beds. The winter of 1906–07 was much more severe than the previous one, and more severe than the average. Minimum temperatures fell below 5° F. on fifteen different days. On twelve days the maximum temperature was under 20° F.; for five successive days in January and

TABLE XII—Maximum and Minimum Daily Temperatures for Winter Months

(Degrees Fahrenheit)

United States Weather Bureau

Day.	December, 1905,		January, 1906.		February, 1906.		December, 1906.		January, 1907.		February, 1907.	
	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.
1	33	12	41	32	47	34	47	30	50	39	36	21
2	54	26	33	26	38	6	30	12	30	12	42	30
3	61	39	35	22	18	—1	34	11	40	30	41	17
4	39	28	58	35	42	16	16	4	57	38	20	14
5	35	22	43	37	43	15	36	13	42	35	21	10
6	38	25	41	31	20	3	54	29	45	34	20	8
7	46	35	34	27	23	9	48	4	58	40	24	6
8	55	36	25	16	30	9	14	1	41	37	30	16
9	39	36	28	10	36	28	18	13	43	17	26	14
10	37	18	26	9	37	23	40	18	36	12	39	22
11	32	15	35	15	25	12	32	14	45	35	37	7
12	28	20	54	34	38	24	25	9	38	21	17	2
13	44	20	37	33	40	34	35	25	34	22	27	1
14	36	13	37	25	40	36	34	29	34	26	52	26
15	23	8	35	24	36	13	53	33	29	16	37	28
16	32	20	52	33	33	10	49	35	16	1	42	26
17	37	19	42	32	34	18	37	33	9	—4	39	20
18	51	25	44	31	37	27	36	12	35	9	26	16
19	44	33	38	25	45	32	30	10	42	29	37	14
20	47	36	46	24	53	32	44	24	58	34	36	28
21	51	36	68	46	63	42	45	36	34	18	38	14
22	50	40	63	48	48	34	41	33	34	17	19	4
23	41	37	68	53	38	28	34	15	17	—2	13	—2
24	38	28	60	25	48	31	24	12	8	—7	23	0
25	37	24	26	19	42	31	27	20	16	4	34	11
26	41	26	34	20	48	32	31	20	22	13	22	4
27	51	32	48	29	35	19	43	25	19	10	24	6
28	55	36	46	35	20	9	40	34	28	13	26	2
29	58	38	36	15	. .	. .	47	36	29	12	. .	. .
30	45	37	48	25	. .	. .	41	38	32	11	. .	. .
31	44	35	56	37	. .	. .	53	38	27	6	. .	. .

for four successive days in February the maximum temperature was under 25° F. (see Table XII). Yet ice and snow collected only at the outer edge and corners of the filters, leaving the main part of their surface clear. The condition of the beds at their worst is shown in Figures 15 and 16. The Columbus nozzle kept open a slightly larger circle, but built up a higher wall of ice at the periphery, owing to its intermittent action. It is interesting to note in Figure 16 the extra ice formation at the two points where the knife-edge supports of the spraying nozzle slightly diminish the flow of sewage. Both methods of distribution, however, worked excellently under the severe winter conditions, and the effluents remained well up to standard.

The analyses of the effluents from filters A and B are tabulated by quarters in Tables XIII and XIV. Regular analyses were not begun till October, 1905, but special analyses during the first month of operation showed that the beds rapidly attained a normal activity. Three parts per million of nitrates were found after a week's operation in the effluent of filter A and five parts in the effluent of filter B. The effluents were extremely turbid at first, and after a month still showed more oxygen consumed than the sewage. By the middle of October, after six weeks of operation, turbidity and organic matter decreased, and the beds assumed their normal operation.

TABLE XIII — COMPOSITION OF EFFLUENT FROM FILTER A

Quarterly Averages

(Parts per Million)

Year.	Quarter.	Temperature F.	Turbidity.	Sediment.	Suspended Solids.		Organic Nitrogen.		Free ammonia.	Nitrites.	Nitrates.	Oxygen Consumed.		Oxygen dissolved.
					Total.	Fixed.	Total.	Solution.				Total.	Solution.	
1905	4	47	155	85	104	27	7.6	2.9	11.6	1.0	3.8	41	24	7.6
1906	1	39	170	110	130	51	6.0	3.9	12.1	0.5	4.0	40	27	9.3
1906	2	53	300	205	186	88	7.6	3.4	13.2	1.8	4.3	51	29	4.7
1906	3	69	183	108	104	45	8.1	3.6	9.3	2.5	3.9	39	24	4.9
1906	4	52	185	155	88	40	6.1	2.9	9.5	0.6	4.5	36	25	7.7
1907	1	39	210	115	87	28	4.9	2.3	8.9	0.3	5.4	33	22	11.2
1907	2	51	270	170	269	130	8.2	2.7	8.5	0.7	5.2	46	26	10.2
Average	. .	50	200	136	138	58	7.1	3.1	10.4	1.1	4.4	41	25	7.9

TABLE XIV — COMPOSITION OF EFFLUENT FROM FILTER B

Quarterly Averages

(Parts per Million)

Year.	Quarter.	Temperature F.	Turbidity.	Sediment.	Suspended Solids.		Organic Nitrogen.		Free ammonia.	Nitrites.	Nitrates.	Oxygen Consumed.		Oxygen dissolved.
					Total.	Fixed.	Total.	Solution.				Total.	Solution.	
1905	4	45	90	45	72	20	5.6	2.7	12.0	0.9	4.1	31	24	8.0
1906	1	39	130	60	83	38	4.9	3.5	14.3	0.1	4.3	31	22	9.3
1906	2	62	215	140	155	85	4.9	2.3	15.3	1.2	4.4	40	23	4.4
1906	3	60	128	66	62	28	6.4	3.4	11.3	3.3	4.5	35	26	5.4
1906	4	52	120	65	50	22	6.0	2.0	11.5	1.2	4.6	33	26	7.6
1907	1	37	145	65	61	22	3.7	1.8	10.7	0.7	5.0	33	27	10.2
1907	2	52	200	105	198	46	3.2	1.1	13.3	1.0	5.7	38	26	8.2
Average	. .	51	147	78	96	37	5.8	2.4	12.6	1.2	4.7	34	25	7.6

From the Tables VII, VIII, XIII, and XIV and from Figures 5 and 6 it appears that the total amount of suspended solids discharged by each filter was slightly in excess of that received by it. An average value of 135 parts per million in the crude sewage was increased to 138 in filter A, and a value of 81 parts in the septic effluent was increased to 96 parts in filter B. A very interesting seasonal curve is indicated, however. The amount of solid material was less in the effluent than in the applied sewage except in the second quarter of the year. In some quarters, filter A effected a 40 per cent. reduction in suspended solids. In the second quarter of each year, there was an enormous increase of suspended solids. in the effluent to a point in excess of the amount in the applied sewage. This means, of course, that the suspended solids of the sewage are ordinarily held on the bacterial films which cover the stones, while during the second quarter the accumulated material is washed out again into the effluent. We suspected at first that spring rains might account for this phenomenon, but a more careful study showed that the increase of solids in the effluent did not correspond to heavy rainfall, and it appeared from calculation that this factor could not be of great importance. Two inches of rain falling in an hour would double the rate on a 2,000,000-gallon per acre filter, but such a storm rarely occurs in the latitude of Boston. Monthly values for suspended solids and temperature, which have been tabulated in Table XV, show, on the other hand, a suggestive correlation.

TABLE XV — SUSPENDED SOLIDS AND TEMPERATURE OF TRICKLING EFFLUENTS

Month.	Filter A.		Filter B.	
	Temperature F.	Solids.	Temperature F.	Solids.
September, 1905	. .	120	. .	173
October	55	110	54	56
November	46	75	45	39
December	41	97	41	52
January, 1906	40	79	41	80
February	39	122	38	. .
March	37	178	38	88
April	46	195	45	124
May	55	237	55	197
June	61	112	62	121
July	69	142	69	76
August	70	75	69	52
September	. .	. .	. .	. .
October	69	62	69	70
November	48	120	47	53
December	40	94	39	68
January, 1907	39	91	37	74
February	37	62	36	51
March	40	107	39	69
April	44	223	44	59
May	51	319	51	213
June	60	316	61	288

The figures in Table XV are tabulated in graphic form in Figure 12. Both filter effluents show a progressive decrease of solid material from May onward till the early spring, when a sudden increase occurs. The trickling filter evidently has its own definite yearly life cycle, and upon this, rather than upon the applied solids, do the solids in the effluent depend. It seems probable that the bacterial films upon the stones absorb or otherwise hold the solid material which reaches them until by this process of accretion and their own multiplication the mass becomes heavy enough to break away. With Boston sewage a year's supply of solids can thus be stored; but the bacterial multiplication set up, when warm weather begins, serves to upset the equilibrium of the bed, and the solids collected during the preceding ten months are discharged. It will be noted, however, by reference to Figure 12, that solids began to increase in A effluent before the onset of warm weather. In each year its rise preceded that of B by two months. This bed received a considerably larger amount of suspended solids than filter B, and it seems probable that the mere accumulation of material began to cause the detachment of zoöglœa before the spring increase in growth occurred. There is at any rate abundant evidence here that the trickling bed automatically frees itself from suspended solids after accumulation has taken place to a certain extent. This phenomenon will militate strongly against permanent clogging of the trickling bed.

In this connection the nature of the suspended matter is also of significance. In filter A the volatile suspended solids were reduced, on the average, from 91 parts to 80 parts, while fixed suspended solids were increased from 44 parts to 58; in filter B volatile suspended solids fell from 63 parts to 59, while fixed suspended solids rose from 18 parts to 37. The larger increase of fixed suspended solids in the septic side is probably due in part to the precipitation of iron sulphide, a deposit of which was later found on the bottom of the bed. In filter A, however, there is no evidence of such a reaction, and the figures seem to indicate a real conversion of organic matter into some insoluble mineral form. It is apparent to the eye that the suspended matter in the effluent of a trickling filter is quite different from that which goes on at the top. It is in much larger particles, as may be seen by the eye, and it settles much more easily. It is for this reason that the turbidity of both effluents was considerably reduced (200 for filter A against 279 in the sewage, and 147 for filter B against 213 in the septic effluent). Microscopic examination by the Sedgwick-Rafter method (Whipple, 1905), carried out under our direction by Mr. H. S. Spaulding, showed that the finely divided particles of suspended matter, of a size less than five standard units, were reduced from 300,000 per cubic centimeter in crude sewage to 28,000 in trickling effluent, while the larger particles, and particularly the macroscopic particles, were increased in numbers. In appearance the material in the trickling effluent is structureless and somewhat flocculent, resembling the amorphous matter so frequently found in water analysis. In its formation the original suspended matter applied to the filter has evidently been thoroughly worked over. It is somewhat surprising, in view of these facts, that the sum of the suspended matter applied and discharged should tally so nearly.

In organic content both effluents show a marked reduction. The purification per cent., as measured for each of the principal constituents, is shown in Table XVI. The results are remarkably close, with the exception of the total "oxygen-consumed" value for the septic side, which exception may be accounted for by the precipitation of iron compounds already alluded to. In all other cases there was a reduction of about 25 per cent. in total organic matter and 40 per cent. in organic matter in solution.

TABLE XVI — PURIFICATION EFFECTED BY TRICKLING BEDS

Percentages

	ORGANIC NITROGEN.		Free ammonia.	OXYGEN CONSUMED.	
	Total.	In solution.		Total.	In solution.
Filter A	22	47	25	27	42
Filter B	23	44	28	40	43

Figures 7 to 11 show that in general the organic matter in the effluent does not exhibit any marked seasonal variations. There is one striking exception, however, to this rule. From Figure 11 it appears that the total "oxygen-consumed" value rose inordinately in the second quarter of each year. This is evidently correlated with the increased discharge of suspended solids at the same time, and it is clear, since no corresponding increase in nitrogen occurred, that the organic matter which then appeared in the effluent was chiefly carbonaceous. This fact strengthens the possibility that the material discharged from the trickling bed is largely bacterial zoöglœa.

Two other points only demand special mention, the dissolved oxygen in the effluents and their content of mineral nitrogen. Dissolved oxygen was at all times present in both effluents in considerable amounts. It was highest in the first quarter of each year, when bacterial decompositions were checked by the cold, and it was distinctly higher during the second year than the first, owing to the better system of distribution. The values for nitrites and nitrates are plotted in Figure 17. Nitrites were at a maximum in both effluents in the autumn of 1906; since the 1907 curve stops at July 1 it is uncerı whether the same phenomena would have been repeated in that ır. It is apparent, however, that the process of nitrite formation is rkedly favored by high temperatures. The curve for nitrates, on other hand, shows a pretty steady increase from the beginning to end of the experiments, and indicates a progressive improvement nitrifying power. Taking the sum of the organic and ammoniacal rogen in applied liquid and effluent, in comparison with the sum of nitrates and nitrites, it appears that filter A received 23 parts unoxidized nitrogen (organic and ammoniacal) and discharged 17.5 ts of unoxidized and 5.5 parts of oxidized nitrogen, nitrates, and rites. Filter B received 24 parts of unoxidized nitrogen and dis-rged 17.6 parts of unoxidized and 5.9 parts of oxidized nitrogen. ere is, therefore, no evidence of appreciable storage or loss of nitro- in filter A, and filter B shows a difference of only half a part. le XVI shows that the per cent. purification effected by filters A B was approximately equal as regards organic matter. Since the d applied to filter B had undergone preliminary treatment in the ic tank its effluent was, of course, actually superior to that of A. It contained only 96 parts of suspended solids against parts for filter A, 5 parts of organic nitrogen against 7.1 parts, 34 parts of oxygen consumed against 41 parts. Its free ammonia nt was, however, higher, making the sum of the ammonia and ic nitrogen practically the same in both cases (17 6 and 17.5). only significant difference between the effluents lay, therefore, fact that filter A discharged more suspended solids, both fixed olatile, than filter B.

ıese results of chemical analysis are of considerable theoretical ıt as throwing light on the processes which go on in the trickling but, as we have pointed out already, they do not adequately e the practical value of the purification effected. Neither the s of the effluents nor the figures for purification per cent. by themselves suggest that our beds were doing satisfactory The methylene blue test, on the other hand, showed that the s of both filters were for the most part stable, and it is this test urnishes the true criterion of their quality.

methylene blue test was made on each effluent daily from y, 1906, to the end of the experiments, and the results are in Figures 18-*A* and 18-*B*. The black blocks indicate the of decolorization by days up to fourteen days, the maximum observation.

uring the cold months of February and March, 1906, results notably irregular for both effluents. The sample collected on lay might prove stable, while that of the next would decolorize ro or three days.

Vith the beginning of warm weather, which favors harmful putre-re changes, both before and during the process of filtration, both

G. 15.—VIEW OF GRAVITY DISTRIBUTOR UNDER SEVERE WINTER CONDITIONS

ents further deteriorated. Filter B recovered about the end of May, throughout the summer its effluent was generally stable. Filter A ained in poor condition until September, and was uncertain in its n for two months longer. The contrast between these beds gs out very clearly the great delicacy of the methylene blue iod of testing effluents. The analyses for the third quarter of 5 in Tables XIII and XIV show, indeed, a slight superiority for filter B, but it is far too slight to lead to the conclusion that at this time one of the filters was practically a success and the other practically a failure.

With the installation of the new sprinklers in November, 1906, a decided change was manifest. After the middle of December, by which time the filter organism had become adjusted to its new conditions, both effluents were almost constantly stable. Occasionally, particularly during the winter months, and with filter A, a putrescible effluent was obtained, but this was rare. From December 15, 1906, to June 28, 1907, 156 samples of each effluent were examined by the methylene blue test. On the crude sewage side 127 samples were stable for fourteen days, nineteen more were stable for over four days, and ten only became decolorized in four days or less at 20°. The effluent of filter B showed 129 samples stable for fourteen days, sixteen more stable for more than four days, and eleven which were decolorized in four days or less. In other words, on 93 per cent. of the days both effluents were of sufficiently good quality to retain free oxygen for over four days.

Three conditions appear mainly responsible for imperfect operation when it did occur. In the first place, excessively low temperatures evidently impair the activity of the purifying organisms. The break in the curve for both filters on February 28, 1906 (see Fig. 18-*A*), was coincident with a fall from 19° to 9° F. The bad results obtained during the first week of December, 1906, accompanied minimum temperatures of 4° to 9° F. (Fig. 18-*B*). After the new distributing apparatus was in good working order, however, this effect of cold weather was only slight. Thus the severe weather of January 16 to 18, January 23 to 25, February 11 to 13, and February 22 to 24, 1907 (see Table XII), shows only slight effects in Figure 18-*B*.

The second factor affecting the stability of the effluents was the spring discharge of suspended solids. This phenomenon in April, 1906, evidently caused the bad break in the quality of the effluents manifest at that time and indicated in Figure 18-*A*. The interesting point in this connection is, however, that the much larger amount of suspended solids washed out in the second quarter of 1907 (see Fig. 5) produced no corresponding increase in putrescibility; that is, with proper distribution the solids discharged are so well oxidized or otherwise rendered stable that they do not interfere with the stability of the effluent.

The third factor which affects the character of the effluent unfavorably is irregularity in the operation of the filter. In Figure 18-*A* the sample taken after a day's rest (the Monday sample) appears again and again the worst of the week, and during the second year the only serious interference with the work of filter B followed the three days' rest of May 18 to 21. This action was always most marked in cold weather. Whether it is due to the injury of the bacteria in the filter by changed conditions or to the detachment of dried zoöglœa is uncertain. It is clear, however, that resting the filter is detrimental to the effluent, and the results for the second quarter of 1907 show that the necessary removal of accumulated material can be accomplished in a natural way without such unfortunate results. We are therefore led to conclude that extended periods of rest for the recuperation of the bed and drying up of the accumulated organic material are not only unnecessary, but actually detrimental, and that the operation of a trickling filter should be made as constant and uniform as possible.

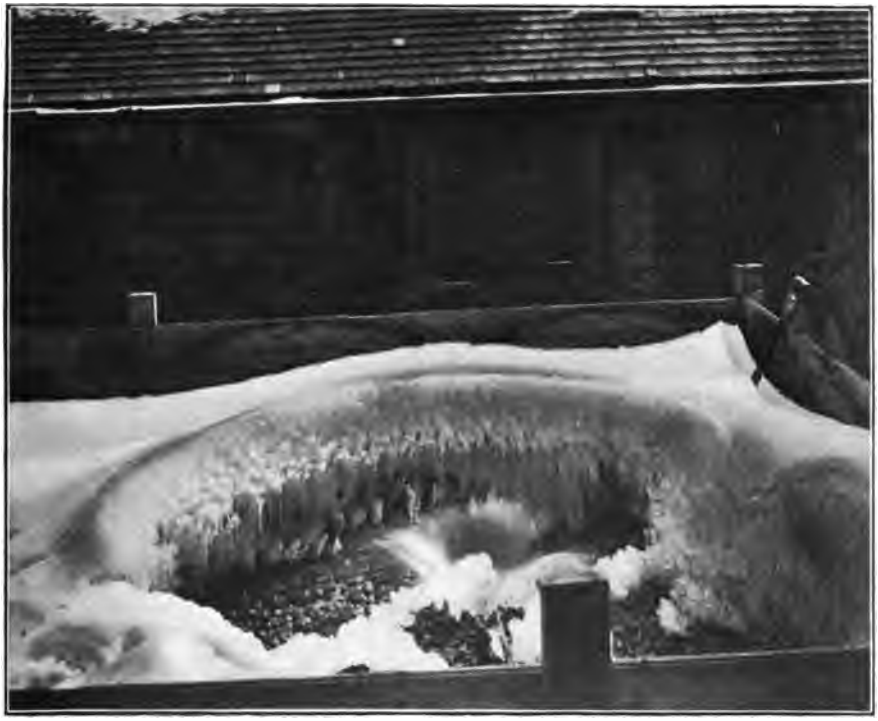


FIG. 16.—VIEW OF COLUMBUS SPRINKLER UNDER SEVERE WINTER CONDITIONS

TABLE XVII — STABILITY OF TRICKLING FILTER EFFLUENTS

Percentage of Samples Decolorized at Each Specified Period

"A" EFFLUENT.										Total samples.
Days		0–2	2–4	4–6	6–8	8–10	10–12	12–14	14+	
Year.	Quarter.									
1906	1	40.5	18.9	2.7	5.4	8.1	0.0	0.0	24.4	37
1906	2	42.6	26.0	20.6	2.7	2.7	2.7	0.0	2.7	73
1906	3	23.7	38.9	15.3	5.1	6.8	0.0	0.0	10.2	59
1906	4 *a*	2.4	9.5	14.3	4.8	9.5	7.1	2.4	50.0	42
1906	4 *b*	2.9	11.4	2.9	8.6	11.4	2.9	8.6	51.3	35
1907	1	2.7	4.1	6.7	2.7	2.7	4.1	8.1	68.9	74
1907	2	4.4	2.9	1.5	0.0	1.5	2.9	1.5	85.3	68

"B" EFFLUENT.										Total samples.
Days		0–2	2–4	4–6	6–8	8–10	10–12	12–14	14+	
Year.	Quarter.									
1906	1	38.8	16.7	5.5	8.4	8.4	5.5	2.8	13.9	36
1906	2	16.9	22.6	26.7	2.8	9.8	1.4	1.4	18.4	71
1906	3	11.9	15.3	10.2	5.1	0.0	0.0	3.4	54.1	59
1906	4 *a*	2.3	0.0	0.0	0.0	4.5	2.3	2.3	88.6	44
1906	4 *b*	14.6	20.4	14.6	11.6	2.8	5.8	5.8	24.4	34
1907	1	4.0	4.0	0.0	2.7	1.3	0.0	4.0	84.0	75
1907	2	5.8	0.0	5.8	5.8	1.4	0.0	0.0	81.2	69

The results of the methylene blue test for each quarter are indicated in Table XVII, samples being divided by their decolorization periods into eight classes, those which decolorized inside of two days, between two and four days, between four and six days, etc., the last class including those which were still stable after fourteen days. The fourth quarter of 1906 is divided so as to differentiate the samples collected before and after the change in the distribution system.

The table shows in each case poor results with the old system of distributors. Filter B treating septic effluent was much better than filter A, however, and showed a steady improvement during this period. When the Columbus sprinkler was installed on filter B a serious temporary derangement followed; but by January, 1907, the filter had become adjusted to the new conditions, and for the last six months yielded effluents, 93 per cent. of which were stable for four days and over 80 per cent. stable for fourteen days. The substitution of a single gravity distributor instead of four on filter A was followed by a steady improvement. Ninety-three per cent. of the samples of A effluent have been stable for four days since January, 1907, and the percentage stable for fourteen days rose from nearly 70 per cent. in the first quarter of the year to 85 per cent. in the second.

Bacterial examinations showed an average of 200,000 bacteria per cubic centimeter in the effluent of filter A and 180,000 in the effluent of filter B. The figures correspond to removals of 83 per cent. and 76 per cent., respectively, figured on the applied liquid. The proportion of acid formers and liquefiers was about the same as in the crude sewage.

At the close of the twenty-one months of experimentation the trickling filters were dug out and carefully examined. Samples of the filtering material were taken at various points; the accumulation between the stones was carefully removed by washing with a brush and the resulting liquor analyzed. In both filters the top of the beds was quite clean. In filter A there was an accumulation of fine material extending from 3 inches down for about a foot just under the four old distributors. This deposit covered four circles, each about a foot and a half in diameter, and was, of course, chargeable to the earlier imperfect process of distribution. In this layer, where the maximum deposit occurred, the extraneous matter amounted to .3 pound per cubic foot, of which 44 per cent. was organic. The remainder of filter A showed no visible accumulation of foreign material, and the underdrains and concrete floor of the bed were as clean as when they were laid. In filter B there was a slight deposit from 6 inches to 18 inches below the surface, amounting to .07 pound per cubic foot, while the remainder of the bed was clean. On the floor of B, however, there was a deposit of sludge half an inch thick, amounting to 4 cubic feet in all (1,720 cubic feet, or 64 cubic yards, per acre). This sludge contained 22 per cent. of water and 1.4 per cent. volatile solids. Qualitative tests showed it to consist of a mixture of finely divided silica with a considerable proportion of sulphide of iron.

From the stone analyses at various depths we have calculated in Table XVIII the total amounts of material stored in each filter and the corresponding values per million gallons. The excess of nitrogen on the septic side is suggestive.

TABLE XVIII — STORAGE OF MATERIAL IN TRICKLING FILTERS

	POUNDS PER FILTER.			POUNDS PER MILLION GALLONS.		
	Fixed solids.	Volatile solids.	Organic nitrogen.	Fixed solids.	Volatile solids.	Organic nitrogen.
Filter A. Crude sewage	36.4	5.5	0.6	14.6	2.2	0.2
Filter B. Septic sewage	5.3	6.6	2.7	2.1	2.6	1.1

In addition to these deposits in the beds the sludge on the bottom of filter B amounted to 697 pounds, or 280 pounds of dry solids, per million gallons. Such material could easily be removed by washing with a hose in a filter with a properly constructed underdrainage system.

VII. SEDIMENTATION OF TRICKLING EFFLUENTS

The effluents from the trickling filter, although stable, contain so much suspended matter as to be unsightly, and under many circumstances further treatment for the removal of suspended solids would be desirable. As shown at many of the English plants and at Columbus (Johnson, 1905), such a removal may be accomplished by simple sedimentation, since the suspended particles in the effluent are sufficiently large to settle out with ease. Since January, 1906, the effluents from filters A and B have been treated in this manner. The tanks used for sedimentation have been described above, and their general construction is indicated in Figures 1 and 2. One of the tanks, photographed before it was put into place, is shown in Figure 19.

The tanks were of conical form, with a central vertical funnel which received the inflowing liquid at the top and discharged it into the body of the tank near its bottom. The supernatant liquid was carried off from the top by two surface channels, and sludge was drawn off by a valve at the bottom. The capacity of each tank was a little under 350 gallons, equivalent to a sedimentation of an hour and three-quarters. Experiments made by Mr. R. R. Patch in a thesis study showed that liquid dyes colored the whole surface of the tank in seven and a half minutes. On the other hand, when a culture of *B. prodigiosus* was poured into the central tube, plates made every five minutes from the outflowing liquid first showed the germ after one hour and gave maximum numbers after an hour and a half.

The character of the effluent from the sedimentation tanks is indicated by the quarterly average analyses in Tables XIX and XX.

TABLE XIX — COMPOSITION OF FILTER A EFFLUENT AFTER SEDIMENTATION

Quarterly Averages

(Parts per Million)

Year.	Quarter.	Turbidity.	Sediment.	Suspended Solids.		Organic Nitrogen.		Free ammonia.	Nitrites.	Nitrates.	Oxygen Consumed.	
				Total.	Fixed.	Total.	Solution.				Total.	Solution.
1906	1	115	45	116	56	4.2	3.9	13.0	0.6	3.9	31	26
1906	2	165	85	53	15	3.8	2.3	13.8	2.1	4.3	36	31
1906	3	100	45	55	20	5.8	4.2	6.3	2.5	4.0	29	25
1906	4	90	50	30	12	4.5	3.0	9.0	0.7	4.5	30	27
1907	1	160	85	50	11	3.0	2.0	9.5	0.5	5.5	29	26
1907	2	135	60	83	29	3.5	2.0	9.5	0.9	5.4	31	25
Average	. .	128	62	65	24	4.1	2.9	10.2	1.2	4.6	31	27

TABLE XX — COMPOSITION OF FILTER B EFFLUENT AFTER SEDIMENTATION

Quarterly Averages

(Parts per Million)

Year.	Quarter.	Turbidity.	Sediment.	Suspended Solids.		Organic Nitrogen.		Free ammonia.	Nitrites.	Nitrates.	Oxygen Consumed.	
				Total.	Fixed.	Total.	Solution.				Total.	Solution.
1906	1	90	30	40	14	3.9	3.2	15.0	0.1	4.4	26	23
1906	2	135	70	63	27	2.8	2.3	14.9	1.2	4.5	30	23
1906	3	76	45	31	11	5.2	4.0	10.7	2.9	4.6	30	28
1906	4	80	35	23	7	4.9	3.5	11.5	1.2	4.4	29	26
1907	1	115	50	34	12	2.5	1.5	10.5	0.8	5.0	31	28
1907	2	140	75	110	37	2.5	1.5	13.0	1.3	6.1	29	27
Average	. .	106	51	50	18	3.6	2.7	12.6	1.3	4.8	29	26

A comparison of the general averages in Tables XIX and XX with the average for the unsettled effluents in Tables XIII and XIV indicates the efficiency of the sedimentation process, and this efficiency has been expressed in purification per cent. in Table XXI.

TABLE XXI — PERCENTAGE PURIFICATION EFFECTED BY SEDIMENTATION

	SUSPENDED SOLIDS.		ORGANIC NITROGEN.		Free ammonia.	OXYGEN CONSUMED.	
	Total.	Fixed.	Suspended.	Soluble.		Suspended.	Soluble.
A effluent	53	59	70	6	2	75	8*
B effluent	48	51	58	13*	0	66	4*

(* = increase.)

Sedimentation for two hours in the conical tanks used in these experiments removed about half the suspended solids present. The removal of suspended nitrogen and carbonaceous matter (oxygen consumed) was in each case greater than the removal of suspended matter in general. Coupled with the increase in soluble nitrogen and carbonaceous matter, this makes it clear that a certain amount of liquefying action went on at the bottom of the tanks. Nitrites and nitrates were unaffected, however, as may be seen by reference to Figure 17. It will be noticed that the per cent. purification effected was always less in the case of the septic effluent. This is, no doubt, because the liquid before treatment contained less gross material to remove. The finer portions of suspended matter naturally settle out less readily.

No important seasonal variations were noted in the action of the sedimentation tanks, except in the case of the organic nitrogen. This constituent, as plotted in Figure 7, was less reduced in the autumn months than at any other time, while at this period (the third quarter of 1906) the soluble organic nitrogen was largely increased. Its value rose from 3.6 to 4.2 in the case of A, and from 3.4 to 4 in the case of B effluent. This was, of course, due to the increased septic action going on in the tanks during the warm season. Bacterial results also indicated the existence of organic activity in the sedimentation tanks. The average number of bacteria in the settled effluent of filter B for the seven months — October, 1906, to April, 1907 — was 260,000 — an increase of 44 per cent. on the effluent entering the sedimentation tank.

The best measure of the efficiency of sedimentation is the methylene blue test, and the decolorization periods for the settled effluents are indicated by the unshaded blocks in Figures 18-*A* and 18-*B*. At times during the winter and spring of 1906 even the settled effluent was putrescible. In general, however, most of the effluents which putrefied in their original state were rendered stable when their solids were reduced by sedimentation. The months of July and August, 1906, furnish a good instance of this. During the whole period forty-nine samples were examined. Of the unsettled effluents forty-five from filter A and twenty from filter B became decolorized inside of fourteen days, but of the corresponding sedimented samples only ten from filter A and nine from filter B became decolorized. After the new distribution system was in working order twenty-nine samples from A and twenty-seven samples from B decolorized in less than fourteen days. Of the corresponding settled samples thirteen from filter A and seventeen from filter B became decolorized.

It must be remembered in interpreting these results that our tanks were not of the best form for securing efficient sedimentation. Their shape was conditioned by the head room at our disposal, and a deeper tank, more of the usual Dortmund shape, would give better results. Furthermore, the pipe for removing sludge from our tanks was so small that sludge could not be drawn off rapidly, and considerable stirring up of the contents of the tank was the result. With tanks of proper design, planned for a full two-hour flow and with proper arrangements for sludge removal, results would undoubtedly be better.

Against the advantages to be gained by the sedimentation of the trickling effluent must be set the new problem of sludge disposal which is introduced. Since September, 1906, this problem has been studied by careful measurement and analysis of the material removed from the sedimentation tanks, and the general results are brought together in Table XXII. The analyses are grouped by four periods, in which the method of removing sludge from the tanks differed as follows: From September to December 3, 1906, and from April 9 to June 29, 1907, twenty-five gallons of sludge and water were drawn from each tank every other day by the valve for removing sludge shown in Figure 2. The three samples for each week were chloroformed, mixed, and analyzed. Reference to Table XXII shows that by this method the liquid removed from the tank was considerable in amount and contained only 10 to 20 per cent. of true sludge. If sludge is removed as often as this it cannot be obtained in very compact form in tanks of such a construction as ours. From December 4, 1906, to January 14, 1907, sludge was removed once a week, the tanks being drawn down to a point 2 feet from the bottom. In this way a more concentrated liquid was obtained. During the third period (January 14 to April 8, 1907) no samples were taken. The sludge was allowed to accumulate, which produced no harmful results in the cold weather; and at the end of the period the tank was drawn

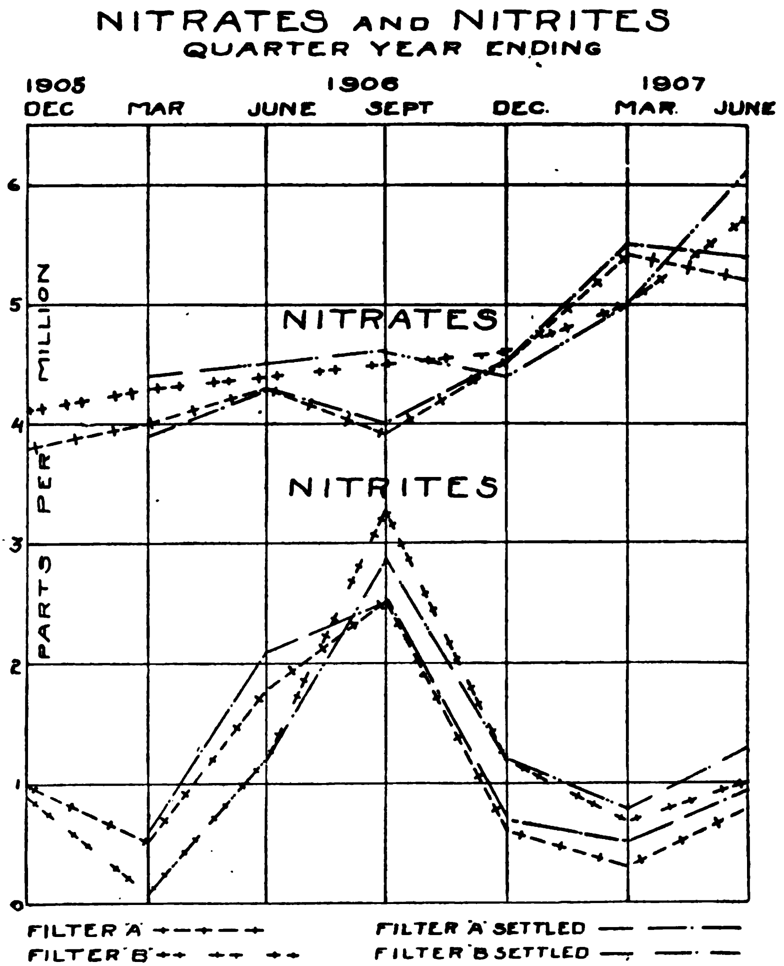
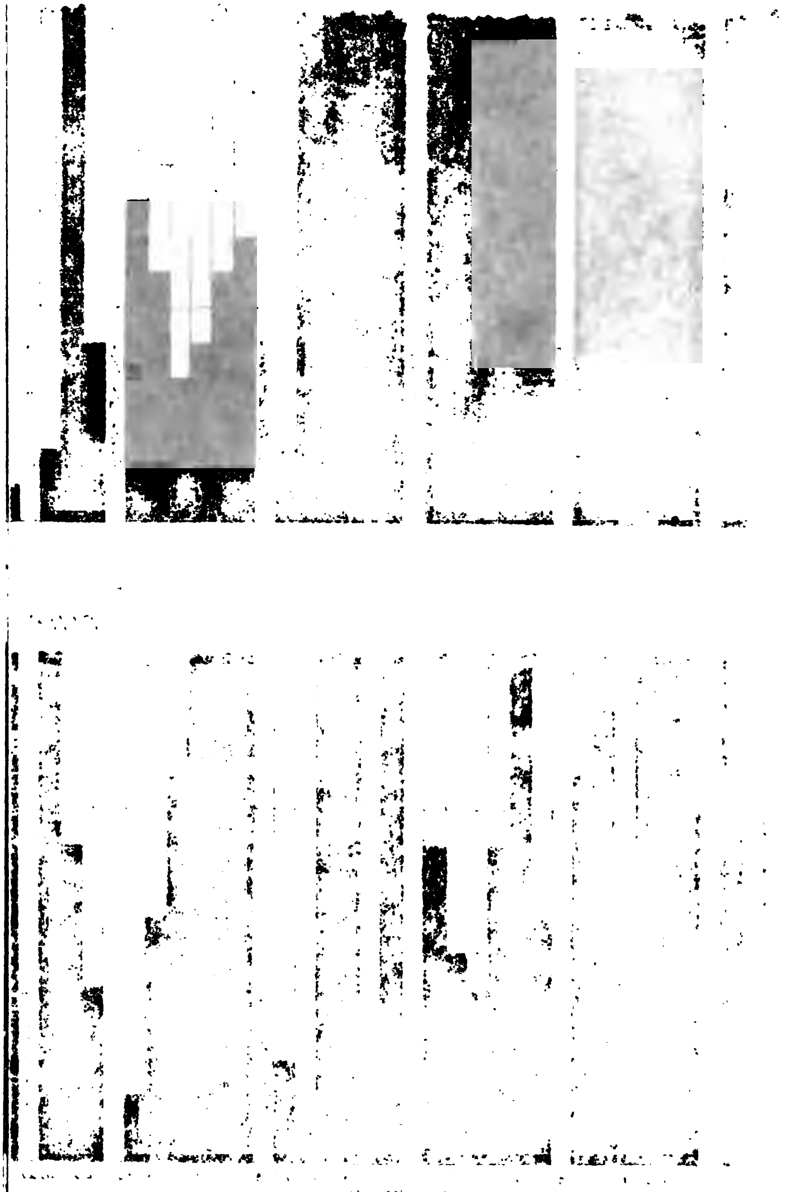
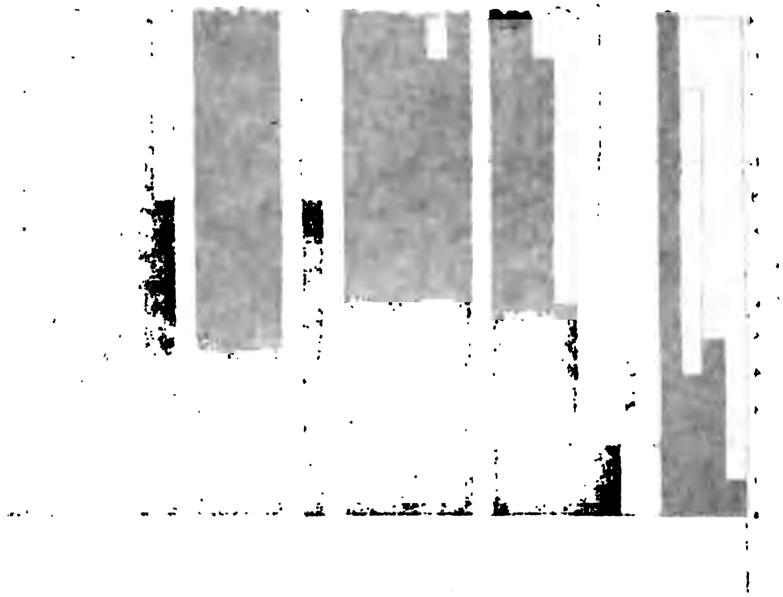


FIG. 17.—QUARTERLY VARIATIONS IN NITROGEN AS NITRATES AND NITRITES; CRUDE AND SETTLED EFFLUENTS

down to a depth of 1 foot 9 inches, and the remaining liquid removed and analyzed.

The cubic yards of wet sludge removed in each period varied for A effluent from .40 to 1.29 and for B effluent from .18 to .73. Per mil-





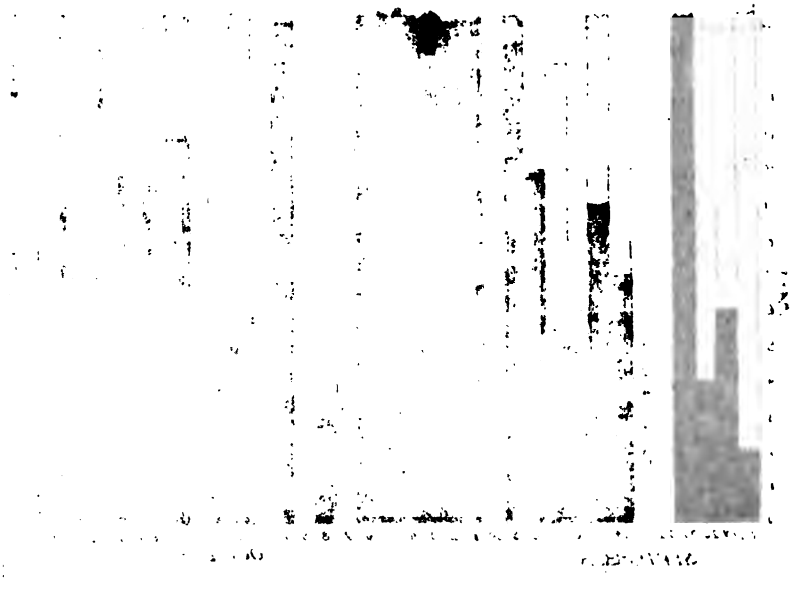


TABLE XXII — AMOUNT AND COMPOSITION OF SLUDGE REMOVED FROM SEDIMENTATION TANKS

Period.	Total gallons sewage passed.	Gallons sludge and water removed.	Per cent. sludge.	Cubic yards wet sludge.	ANALYSIS OF SLUDGE, PER CENT.					POUNDS PER MILLION GALLONS SEWAGE.			
					Solids.		Organic nitrogen.	Carbon.	Specific gravity.	Solids.		Organic nitrogen.	Carbon.
					Loss.	Fixed.				Loss.	Fixed.		
A.													
September—December 3 . . .	276,000	825	15.5	0.636	4.07	5.07	0.195	0.45	1.049	155.0	193.0	7.40	17.10
December 4—January 14 . . .	151,800	90	90.5	0.405	4.32	5.84	0.190	0.45	1.049	184.5	257.0	8.35	19.80
January 15—April 8	285,200	490	38.0	0.925	3.72	3.81	0.150	0.35	1.034	200.0	206.0	8.06	18.90
April 9—June 29	246,000	900	29.0	1.290	4.22	5.12	0.215	0.30	1.028	361.0	439.0	18.40	25.80
B.													
September—December 3 . . .	276,000	825	10.2	0.420	3.81	3.85	0.223	0.45	1.045	96.5	97.5	5.70	11.40
December 4—January 14 . . .	151,800	90	42.0	0.185	3.99	4.43	0.195	0.38	1.042	82.6	92.1	4.05	7.89
January 15—April 8	285,200	490	30.0	0.730	3.30	3.56	0.220	0.20	1.036	138.0	151.0	9.40	8.50
April 9—June 29	246,000	900	16.0	0.712	4.53	3.39	0.205	0.33	1.025	220.0	165.0	9.96	16.10

lion gallons, the cubic yards of sludge removed were, for the four successive periods, 2.3, 2.7, 3.2, and 5.7 for filter A, and 1.5, 1.2, 2.6, and 2.9 for filter B. The increased values in the last two periods are due to the increased suspended solids in the effluents from the trickling beds during the spring rains. Taking the whole period from September to June, there were removed from the sedimentation tanks, per million gallons of sewage passed, on the crude sewage side, 3.40 cubic yards of sludge, with 237 pounds of volatile solids, 281 pounds of fixed solids, 11.2 of organic nitrogen, and 20.8 of organic carbon. On the septic side, per million gallons of sewage passed, there were 2.14 cubic yards of sludge, containing 136 pounds of volatile solids, 126 pounds of fixed solids, 7.5 pounds of organic nitrogen, and 11 pounds of organic carbon.

The volume of this sludge is evidently considerable. In composition it is naturally not so putrescible as its analysis would indicate, since it consists largely of the humus-like compounds formed in the trickling bed. Nevertheless, in concentrated form it is not generally stable, as shown by the fermentations already alluded to as going on in the sedimentation tanks. Mixed with the rest of the effluent these solids do not putrefy, as shown by the methylene blue tests of the unsettled effluent, and by the fact that nitrites and nitrates showed no diminution in the sedimentation tanks. By itself, however, the sludge will generally putrefy, and if it is removed by sedimentation some means must be found for its disposal. We attempted to purify it by straining through coke without success. Coarse coke allowed it to pass freely and fine coke clogged too rapidly to be of practical use. It will probably be found best under actual working conditions to discharge the tank contents on to sludge beds and dig the resulting deposit into the land. From the inconsiderable putrescibility of the sludge, this method of disposal could be accomplished with ease. It will, however, be a local question in each individual case whether the improvement effected in the trickling effluent by sedimentation compensates for the increased cost of settling and of sludge disposal.

VIII. DISINFECTION OF TRICKLING EFFLUENTS

The trickling filter furnishes the best mechanism now available for accomplishing the primary object of sewage purification — the oxidation, or otherwise rendering stable, of organic matter. Often this is enough; but where shellfish industries or water supplies are menaced the bacteria too must be taken into account. For bacterial removal the rapid processes of sewage treatment are entirely inadequate. Comparing the effluent of filter B with the crude sewage, it is apparent that septic tank and trickling bed combined reduced the total number of bacteria from 1,200,000 to 180,000 — a diminution of 85 per cent. Tests for *B. coli* during the summer of 1906 gave the results indicated in Table XXIII, the figures representing the average of thirty-five determinations.

TABLE XXIII — BACTERIA IN SEWAGE AND SEPTIC AND TRICKLING EFFLUENTS

	Bacteria per c.c. Lactose agar at 37°.	*B. coli.* Positive tests* in $\frac{1}{1000000}$ c.c.
Sewage	1,300,000	65 per cent.
Trickling filter receiving sewage	750,000	35 per cent.
Septic effluent	1,650,000	66 per cent.
Trickling filter receiving septic effluent	750,000	35 per cent.

* By Jackson bile test.

Evidently during the warm weather there was a multiplication of bacteria in the septic tank, and the removal of total bacteria and of gas-forming organisms by the whole process was under 50 per cent.

These results are as good as can be expected from filters of coarse material, but they do not represent satisfactory bacterial purification. For this reason the Baltimore experts (Baltimore, 1906) suggested secondary filtration through sand for the effluents from their projected trickling filters, in order to avoid possible danger to the shellfish industry of Chesapeake Bay. The Board of Advisory Engineers estimated the cost of works for the complete treatment of 75,000,000 gallons of sewage per day at $3,283,250, of which sum $1,040,750, or over 31 per cent., was for the supplementary treatment on sand filters. The annual cost of operation was estimated at $115,500, of which $55,000, or 48 per cent., was for the supplementary treatment (Baltimore, 1906). This gives some idea of the price which it is necessary to pay for effluents of bacterial purity over and above the cost of rendering the sewage stable and non-putrefactive.

Fortunately a far cheaper and equally efficient method of removing bacteria is now at hand. The application of chemicals as a method of sewage purification — that is, as a method for the removal of organic material — has not proved successful except in rare instances, with sewage of peculiar composition. When organic stability has been attained, however, and bacterial removal only is desired, the problem becomes a wholly different one. Disinfection of sewage and effluents has been studied by various English observers, most exhaustively by Rideal (1905). The latter observer, in an extensive series of experiments at Guildford, studied the action upon raw and septic sewage, and upon the effluents from contact beds, of hypochlorites produced electrolytically from sea water by the oxychloride process. Great reductions in bacteria were obtained with 30 to 70 parts per million of available chlorine for the crude sewage, 25 to 44 parts for septic sewage, and .5 to 20 parts for contact effluents of varying grades of purity. The removal of *B. coli* was all that could be desired. Examinations of the few organisms remaining in the sewages and effluents after treatment showed them to be largely organisms of the hay bacillus group, aërobic, spore-forming bacteria. Absolute sterilization, however, required very high concentration of chlorine. Since American sewages are weaker than European, and trickling effluents more highly purified than those of ordinary contact systems, the outlook for disinfection seemed more favorable under our conditions, for the less organic matter present the less chlorine will be absorbed and prevented from exerting its disinfectant action. Phelps and Carpenter (1906), in a study of the problem on a laboratory scale, obtained the results indicated in Table XXIV.

TABLE XXIV—DISINFECTION OF TRICKLING EFFLUENT BY BLEACHING POWDER

(Phelps and Carpenter, 1906)

Experiment No.	Available chlorine. Parts per million.	ORGANISMS KILLED. PER CENT. OF INITIAL NUMBERS.				
		Thirty minutes.	One hour.	Two hours.	Five hours.	Twenty hours.
I	3.5	98.7	. .	97.8	99.6	99.9
	10.6	98.9	. .	98.9	99.7	99.8
	55.0	99.9	. .	99.9	99.9	99.9
II	25.0	. .	99.8	99.9	. .	. .
	37.0	. .	99.94	. .	. .	. .
	100.0	. .	99.97	. .	. .	. .
III	2.0	. .	99.8	99.9	. .	. .
	5.0	. .	99.85	99.9	. .	. .
IV	0.25	. .	65.6	70.9	. .	. .
	0.50	. .	89.2	94.2	. .	. .
V	1.0	. .	81.5	99.1	. .	. .
	1.5	. .	99.6	99.9	. .	. .
VI	5.0	. .	98.2	98.2	. .	. .
	10.0	. .	. .	99.6	. .	. .

It was evident from these experiments that complete sterilization was a practical impossibility. Neither the addition of large amounts of bleaching powder, up to 100 parts of available chlorine per million, nor the storage of the effluent for periods of time up to twenty hours proved entirely effectual. On the other hand, a comparatively low concentration of chlorine applied for two hours gave a very large reduction, the residual organisms, as in Rideal's experiments, being almost entirely saprophytic spore formers. Sterilization is not feasible, but disinfection is.

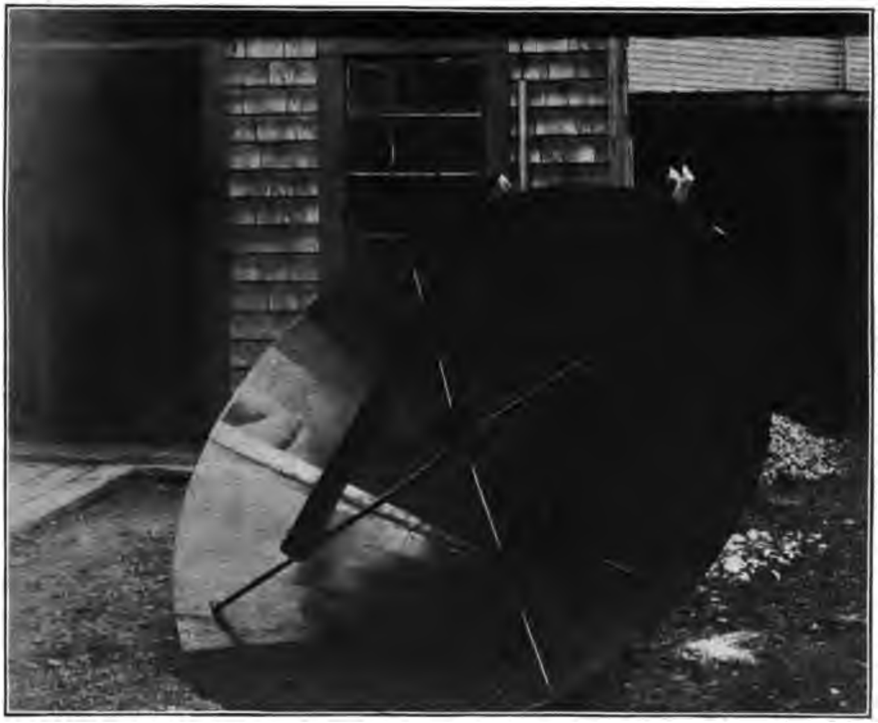


FIG. 19.—VIEW OF SEDIMENTATION TANK BEFORE IT WAS PUT IN PLACE

It is important to maintain the distinction between absolute sterilization and a reasonable percentage purification such as is obtained in water filtration. A neglect of this discrimination has led to the use of excessive quantities of lime in certain recent German experiments (Schumacher, 1905; Kranepuhl, 1907; Kurpjuweit, 1907). The character of the individual sewage, however, will of course materially affect the results obtained in a given case. A good efficiency has been reported in recent experiments on the disinfection of septic effluent at Bengal, India. On the other hand, Kellerman, Pratt, and Kimberly (1907) found considerable amounts of chloride of lime necessary for the treatment of septic and contact effluents in Ohio.

In August, 1906, we began to treat the effluent from filter A by adding chloride of lime solution from a small orifice box to the effluent as it flowed into the sedimentation tank. The solution was mixed once a day in a fifty-gallon barrel in such strength that when added to the effluent at the rate of two gallons per hour it would supply the effluent with about five parts per million of available chlorine. It was not possible on such a small scale to regulate the flow accurately, and there was some variation in the amount of available chlorine added to the effluent. This value was determined each day when the samples were taken for bacterial examination. It averaged about five parts per million, and the variation was no more than may be expected in the case of a larger plant with less careful supervision. Samples were examined before the disinfectant was added and at the outflow from the tank after a little less than two hours' contact.

TABLE XXV — EFFICIENCY OF DISINFECTION OF TRICKLING EFFLUENT WITH CHLORIDE OF LIME

(Available Chlorine, Five Parts per Million)

Date.	ORGANISMS PER CUBIC CENTIMETER.		GAS PRODUCTION IN BILE BROTH.	
	Before treatment.	After treatment.	Before treatment. Dilution, $\frac{1}{1000000}$ c.c.	After treatment. Dilution, 1 c.c.
August, 1906				
11	270,000	69	+ 0	+ 0
13	630,000	41	0 0	+ 0
14	135,000	406	+ +	+ 0
15	230,000	21	0 0	0 0
16	250,000	37	+ 0	0 0
18	110,000	40	0 0	+ 0
20	90,000	54	+ 0	0 0
21	220,000	22	0 0	0 0
23	. .	. .	+ 0	0 0
Average	240,000	86	33 per cent.	22 per cent.
Average removal		99.96 per cent.		99.993 per cent.

The results of bacterial examinations on nine different days during the period, as indicated in Table XXV, show a 99.96 per cent. removal of total organisms. With Jackson's bile test positive results were obtained in 33 per cent. of the tests made upon the untreated effluent at a dilution of one millionth, indicating that the organism was present on the average to the extent of about 330,000 per cubic centimeter. After treatment, positive tests were obtained in 22 per cent. of 1 c.c. samples, indicating that on the average there was an organism present in 5 c.c. of effluent. Upon this basis the average removal of fermenting organisms (presumably *B. coli*) was 99.993 per cent.

These results were so favorable as to indicate the entire feasibility of disinfecting the effluent from trickling beds by means of chlorine. Five parts of chlorine, the amount necessary to secure practical disinfection, could be applied at a cost of $1.50 per million gallons of sewage. We desired, however, to compare the efficiency of other disinfecting agents. Johnson and Copeland (1905) obtained good results with copper sulphate, but used amounts of copper and periods of storage which would make the cost of the process prohibitive. When our filters were put in operation after their rest in September, 1906, experiments were begun with copper sulphate. One part of copperas sulphate per million was added to A effluent from October 13 to November 19, and from November 20 to December 2 two parts were used in the same way. During the first period the average of examinations on fourteen days showed 230,000 bacteria per cubic centimeter before treatment and 34,000 afterward — a reduction of 35 per cent. During the second period (with two parts of copper per million) the average of examinations on seven days showed 240,000 bacteria per cubic centimeter before treatment and 18,000 after — a reduction of 92.5 per cent. Evidently copper is inferior to chlorine for the treatment of this effluent, since the cost of one part of copper is about equal to that of five parts of chlorine.

During January and February, 1907, no disinfectant was added to the effluent from filter A. Simple sedimentation showed for the average of fifteen determinations a reduction from 140,000 bacteria to 42,000 — a diminution of 70 per cent. At higher temperatures, when multiplication takes place in the tank, no such reduction would, of course, be observed. As noted above, the tank receiving B effluent showed a considerable net increase of bacteria in the period from October to April.

March 1 experiments were begun with a third disinfectant, sodium benzoate, which has been suggested for use in this connection. In March and April, 0.8 part per million gallons was added, and in May the quantity was doubled. For the first period the average of tests on twenty-six different days showed 140,000 bacteria per cubic centimeter before treatment and 54,000 afterward — a reduction of 62 per cent. For the second period the average of five determinations showed 370,000 bacteria per cubic centimeter before treatment and 84,000 afterward — a reduction of 78 per cent. Evidently sodium benzoate is not an economical disinfectant, since the application of one part per million gallons would cost $1 per million gallons.

There is one important factor to be taken into account in interpreting these results, that of temperature. The action of disinfecting agents follows the velocity laws of other chemical reactions, and rises with an increase of temperature. The chlorine work was, therefore, carried out under somewhat more favorable conditions than the experiments with copper sulphate and sodium benzoate. The difference between chloride of lime and the other agents tested was so great, however, as to leave little question of the practical superiority of chlorine for the treatment of our effluent. Recent work carried out by one of us for the New Jersey State Sewerage Commission at Red Bank, New Jersey, has shown that chlorine disinfection is entirely feasible at a low cost and on a practical scale.

IX. CONCLUSIONS AS TO THE BEST METHODS OF TREATING BOSTON SEWAGE, WITH AN ESTIMATE OF ITS APPROXIMATE COST

1. *Trickling Beds.* — The main result of this investigation has been to show the feasibility of treating Boston sewage on trickling beds so as to secure organic stability. In the experiments conducted, the filters were operated at a rate of about two million gallons per acre per day, which would call for fifty acres of stone beds for the treatment of the sewage now discharged at Moon Island. A comparison with the 133 acres of contact beds or 1,000 acres of sand beds, which would be necessary for other processes, indicates clearly that for this city the trickling bed offers the most practical method of treatment.

We have found that with good distribution a trickling bed 8 feet deep will operate successfully at all seasons, under local weather conditions. It removes about half the soluble organic matter, yielding an effluent which is somewhat turbid, but stable and well oxygenated. The organic matter present has been so worked over and purified by the bacteria in the filter as to be non-putrescible. Judged by the methylene blue reduction test 93 per cent. of the samples of the effluent are of such stability as to undergo no putrefactive change when kept closed up from the air for four days. Under ordinary conditions of discharge into open water such an effluent would be entirely unobjectionable.

The proper distribution on trickling beds can be attained either by the use of fixed sprinkler heads of the Columbus type, so arranged as to discharge intermittently at frequent intervals, or by the use of the splashing gravity distributors designed at the Experiment Station for this purpose.

With good distribution the trickling beds show no appreciable tendency to clog. During the greater part of the year solid matter accumulates on the surface of the stones throughout the bed, but when this storage reaches a certain point, usually in the early spring, the solids break away and come off in the effluent in a stable condition. In a period covering two years the total amount of solid matter coming off balanced that going on. The filtering material at the end of the experiments was in excellent condition and showed no storage of nitrogen.

Our results point strongly to the advantage of operating trickling beds under conditions as uniform as possible. Resting periods proved distinctly detrimental to the work of the beds, and constant operation is to be recommended rather than any process which involves alternate working and resting periods.

2. *Septic Tanks.* — It appears from our experiments that Boston sewage may be treated in the septic tank with excellent results, and that a period of seven hours is a sufficiently long one. Thus operated an open tank will remove 40 per cent. of the total suspended solids and 60 per cent. of the fixed suspended solids; its effluent shows a decrease of about 25 per cent. in organic nitrogen in solution and a corresponding increase in free ammonia. The septic action on the stored solids is an active one, four-fifths of the organic solids deposited disappearing in solution or as gas. Fixed solids gradually accumulate so as to render it probable that tanks would require cleaning about once in two years.

On the whole, our experiments indicate that the septic tank need not be used at all in the treatment of Boston sewage. Since November, 1906, when the distribution system was put in order, crude sewage has been treated on one of our trickling beds with perfect success. The effluent from this filter was less frequently putrescible than that from the bed which received septic effluent. The filter taking septic effluent showed a deposit on its floor, due to secondary reducing changes, which was absent from the crude sewage bed. Furthermore, the absence of the odors produced by spraying septic sewage is an advantage of considerable moment in favor of the process of treating fresh sewage. Combined with the saving of the cost of tanks (in the neighborhood of $250,000), these arguments seem to indicate the treatment of crude sewage directly on trickling filters as most desirable. Modern devices for ensuring a thorough preliminary screening should, however, be installed.

3. *Sedimentation of Trickling Effluents.* — The suspended solids which appear in the trickling effluent, though inoffensive, are unsightly, and in many locations might require removal. By a sedimentation of two hours we have found it possible to remove about half the suspended solids. This clarification was accompanied by an improvement in stability.

In the case of Boston the currents of the harbor would be amply competent to care for the solid matter discharged if that matter were of an inoffensive and non-putrescible nature. Experience with the system at present in use has indicated this quite clearly. For a comparatively slight improvement in stability it does not appear to us justifiable to go to the expense of installing secondary sedimentation tanks. The sludge accumulating in such tanks would amount to two or three cubic yards per million gallons of effluent — a serious problem in itself. We are therefore of the opinion that the effluent from the trickling beds may best be discharged directly into the harbor as it comes from the beds. A stable effluent under such conditions could cause no nuisance, and if a submerged discharge were provided its presence would scarcely be detected.

4. *Disinfection of Trickling Effluent.* — The problem of bacterial purification still remains to be considered, since the trickling bed produces organic stability without destroying pathogenic bacteria. In the case of Boston Harbor, with its large contiguous population, its bathers, and its shellfish industry, this aspect is an important one. The experiments carried out during the last two years have made it clear that

the effluents from trickling beds may be so purified bacterially by disinfection with chloride of lime as to be of much better quality than the present streams entering Boston Harbor. This bacterial purification requires about five parts of available chlorine per million gallons, and the cost of treatment would be within moderate limits.

The process of disinfection with chlorine can be applied to crude sewage as well as to trickling effluent, although experiments carried out at the station indicate that about double the amount of chlorine is needed, on account of the reducing action of the organic matter in the sewage. Pending the construction of a trickling filter plant for the treatment of the organic matter in Boston sewage, it might well be purified bacterially by this process at the present Moon Island outfall.

5. *General Plan for the Treatment of the Sewage of the South Metropolitan District.* — The sewage outfall of the south Metropolitan district at Moon Island is the one which threatens most seriously to menace the purity of Boston Harbor, and it is this sewage which will certainly first require some different method of treatment. We have, therefore, considered in a general way the practical problem of dealing with it in the light of the results of our experiments.

The most convenient location for a trickling filter area would be at the calf pasture in Dorchester, near the present pumping station. This is objectionable, however, on account of its proximity to the thickly settled portion of Dorchester. Furthermore, the necessity for excavating about 10 feet of mud and refilling in its place would greatly increase the cost of construction at this point. The same objections apply to certain waste areas on the Neponset marshes which suggested themselves as possibly available. The headland of Squantum would offer an ideal opportunity for building trickling beds, but the difficulties of obtaining land in another town militate against the use of this site.

The southern portion of Thompson's Island would furnish a location free from all the objections to which the other sites are open. On an embankment 1,500 feet long the sewage could be carried from Squantum across to the island, the effluent flowing back along the same embankment to the existing outfall sewer. The pumping station at Dorchester and the tanks and outlet at Moon Island could thus be used without substantial changes. We have made preliminary estimates of the cost of building fifty acres of trickling beds, 8 feet deep, and equipped with the gravity distribution system, and are of the opinion that the cost, including the embankment with its two sewers, an efficient grit chamber, a reasonable purchase price for the necessary land, grading, stone filling brought to the island by water, concrete construction, and sprinklers, would be in the neighborhood of $1,800,000.

If this capital sum were borrowed at 5 per cent. on a twenty-five year loan the annual expense for interest and sinking fund would be $126,800, paying off the entire cost in the twenty-five year period. As a matter of fact, we see no reason to suppose that at the end of this time the plant would not be good for another twenty-five years without substantial reconstruction. The cost of operation, including extra pumping and supervision of screens and filters, would amount to $70,000 a year, bringing the total cost to about $200,000 a year, or $5.50 per million gallons of sewage treated.[1]

The effluent from the trickling beds, wherever situated, could be further bacterially purified by disinfection with chloride of lime, at a cost of approximately $1.50 per million gallons, or $55,000 annually.

Pending the construction of filters for the removal of putrescible organic matter from Boston sewage, if it should seem desirable to secure bacterial purification, this may be effected by direct treatment of the crude sewage with chloride of lime, which could probably be done for $3 per million gallons, or $110,000 annually.

Experiments are now in progress at the Experiment Station to test the practicability of higher rates of filtration and shallower beds than those used in the experiments on which these calculations are based, as well as on the treatment of sewage and effluents by electrolytically produced chlorine. It is hoped that these experiments may lead to a material reduction in the estimated cost of the purification processes. It seems clear, however, that the combination of trickling filters and chemical disinfection will solve the Boston sewage problem satisfactorily; and in the light of present knowledge these two methods are the most efficient and economical available for the purpose.

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[1] The authors desire to express their thanks to Mr. W. S. Johnson for assistance and advice in the preparation of these preliminary estimates.

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CONTRIBUTION FROM THE SANITARY RESEARCH LABORATORY AND SEWAGE EXPERIMENT STATION

A METHOD FOR TESTING AND COMPARING SEWAGE SPRINKLERS[1]

BY EARLE B. PHELPS

IN the application of sewage to the surface of filters of the trickling or sprinkling type, evenness of distribution is an essential factor. This is more especially true in the case of filters whose material is of so coarse a nature that there is little or no lateral movement of the sewage within the bed. If on such a filter one-third of the surface is receiving two-thirds of the applied sewage, it is quite evident that the rate of filtration on that area is twice the mean rate. This rate would largely determine the character of the effluent, and an effluent of the same character could be obtained at twice the rate of filtration if the sewage could be distributed in a perfectly even manner over the whole area.

To accomplish this distribution over the surface of coarse-grained filters, many types of distributor have been devised. Among the more important of these may be mentioned: (1) Systems of fixed troughs, designed either to overflow or to discharge through notches or orifices; (2) delivery arms rotating over circular beds, operated either by the flow of the sewage or by power, distributing through nozzles or by overflowing; (3) large overflow troughs moved laterally back and forth over the bed; and (4) fixed sprinkler heads (pressure sprinklers) discharging upward under pressure, or discharging downward (gravity sprinklers) upon sprinkling or spattering disks. The distribution effected by these various methods is necessarily more or less uneven, and the efficiency of a given system may be defined as the evenness of the resulting distribution, a perfect distribution being one in which each small unit of area is receiving its proportionate share of the sewage. Such a condition is approached, in theory at least, by well-designed distributors of the second and third types enumerated. Their efficiencies depend upon proper leveling, unobstructed openings, and other details of design and operation which need not be gone into. Distributors of the first type mentioned cannot give perfect distribution. They distribute in lines, and as these lines are brought nearer and nearer together the distribution becomes more nearly perfect. In the Stoddart distributor the limit is reached and almost perfect distribution achieved by completely covering the filter with corrugated sheets of metal, the individual troughs being in actual contact. It is with the last group mentioned — pressure and gravity sprinklers — that the present discussion has to deal. Such sprinklers are fixed at definite points at or above the surface of the bed, and are designed to throw the sewage in a spray or spatter over the intermediate areas. The efficiency of their distribution under given conditions of head is determined entirely by their design, and the study of this efficiency is one which lends itself readily to careful experimental methods. The distribution is obviously not a perfect one, if for no other reason than that it is circular, and there must either be overlapping of the circles or intermediate non-wetted spaces, or both. Within each circle, moreover, there is generally a tendency to the formation of concentric rings of greater or less flow than the average, so that if small units of area are studied there will be found a variation in rate of flow along a radius. The definition of perfect distribution is one which is easily grasped, but the conception of a quantitative measure of the unevenness of the distribution is not so simple, and no attempt has heretofore been made to work out such a measure. Obviously the distribution is not measured by the wetted surface, although that expression is the best one which has thus far been used to describe the efficiency of a sprinkler. In the course of an investigation of pressure and gravity sprinklers now being made at this station, the need of some quantitative measure of the results was at once manifest. A mathematical analysis of the conditions involved was made, resulting in the definition of a "coefficient of distribution," which shows the mathematical relation between any observed condition of distribution and that condition which has been defined as perfect. In view of the ever increasing importance which sprinkling filters are assuming in this country, it is thought wise to publish this preliminary description of the methods of testing employed and of the mathematical analysis of the results. A graphical representation of the conditions of distribution is first worked out, and from it the definition of the "distribution coefficient" is derived.

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[1] Reprinted from Engineering News of October 18, 1906, LVI, 410–411.

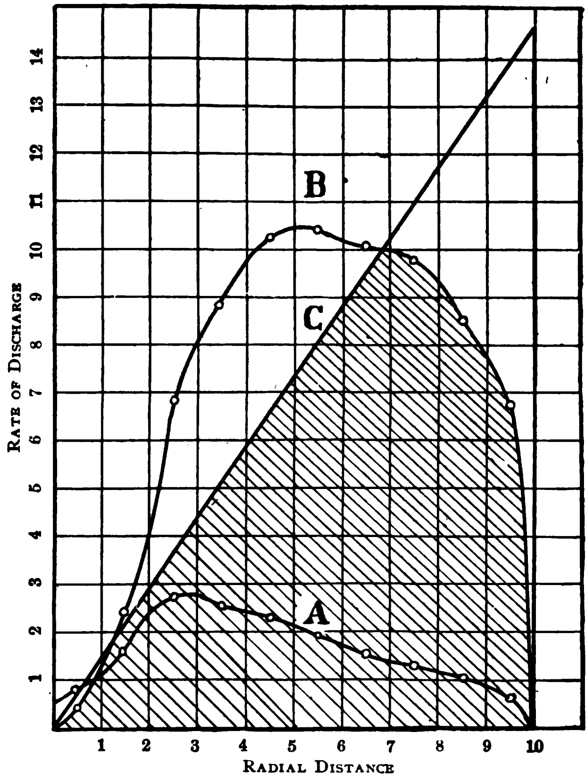
For the purposes of this problem let us assume that a pressure sprinkler is shooting up a spray of sewage, fountain-wise, the sewage falling over a circular area. Along any radius let us place a line of small, square collecting tanks of known areas, which we may designate the unit area. Let it be assumed, further, that the distribution along this radius is the mean of that along all radii. The variations along the various radii will be small, and the assumption may be completely realized, as is being done in our experiments, by revolving the line of collecting tanks about the centre during the experiment. After running a suitable length of time the sewage is shut off and the amount collected in each small tank is measured. In the accompanying table a set of actual results is reproduced for the sake of illustration.

TABLE SHOWING RESULTS OF OBSERVATIONS OF DISTRIBUTION EFFECTED BY SEWAGE SPRINKLER

Tank No.	Radial distance (D).	Sewage quantity (Q).	D × Q.	Excessive Quantities (Ex. Q).	D × Ex. Q.
1	0.5	0.8	0.4	. .	. .
2	1.5	1.6	2.4	0.14	0.21
3	2.5	2.7	6.8	1.24	3.10
4	3.5	2.5	8.8	1.04	3.65
5	4.5	2.3	10.3	0.84	3.78
6	5.5	1.9	10.4	0.44	2.43
7	6.5	1.5	10.0	0.04	0.26
8	7.5	1.3	9.8	. .	. .
9	8.5	1.0	8.5	. .	. .
10	9.5	0.6	5.7	. .	. .
Totals	50.0		73.1		13.43

$73.1 \div 50.0 = 1.46 = M.$

In the first column are given the numbers of the collecting tanks, No. 1 being at the centre. In column 2, headed D, are given the distances from the centre of the sprinkler to the centres of the corresponding tanks, the side of the tank being taken as unit distance. In the third column, Q, are given the amounts of sewage measured in each tank at the end of the test. Any unit of volume may be employed. In this case the depths in the measuring tanks are used, these being proportional to the volumes. Now if these quantities, Q, are plotted as ordinates against the corresponding D values as abscissæ, we obtain a curve showing the relative distribution of the sewage along the radius. This curve is shown at A in the diagram. It shows the rate of discharge per unit area at any point whose distance from the centre is known, and will be known as the curve of radial distribution. Since the radial distribution measured was assumed to be the mean of all radial distributions, the ordinate of this curve at any given distance from the



centre shows the rate of discharge at all points on a circumference described about the centre at the given distance from it. Hence to obtain a curve showing the distribution on the whole circle, *i. e.*, on an infinite series of successive circumferences, it is necessary to multiply the ordinate for each point on the radius by the length of the corresponding circumference, or what amounts to the same thing, relatively, by the radial distance of the point in question. In column 4, D × Q in the table, are given the figures resulting from this operation. These quantities represent the relative rates of discharge of the sewage upon successive concentric rings of the circle, the width of the rings being unity and their distances from the centre being the corresponding D. If now a new curve be plotted with these products just obtained as ordinates and the corresponding D values as abscissæ, the resulting curve is the curve of distribution over the wetted area (curve B in the figure).

From its construction this curve has the following properties: the ordinate at any point shows the relative rate of discharge of the sewage upon a circumference at the corresponding distance from the centre; the area of any vertical strip shows the relative total discharge upon the corresponding ring of the wetted circle; and the total area under the curve represents the total discharge from the sprinkler.

It will next be necessary to construct the curve of perfect distribution. This curve must conform to the fact that the discharge upon all unit areas is the same, and hence that discharges upon all circumferences are directly proportional to their lengths, *i. e.*, to their radii. Obviously, then, the curve is a straight line passing through the origin. Its slope is determined graphically by the fact that the area under the curve represents the total discharge from the sprinkler, and hence equals the area under the distribution curve already drawn. The latter area may be determined by means of a planimeter. The base of the triangle is known, and from that and the area its altitude may be computed. It may be calculated more readily but less accurately by adding up the column Q × D and dividing by the radius of the wetted circle (10). This curve is shown in the figure as curve C.

We now have the curve of actual distribution and the curve of perfect distribution. There remains to be derived a mathematical expression of the relation between these two curves. The curves have a common area, which is shaded in the diagram. From what has preceded it will be apparent that the more perfect the distribution the more nearly will the two curves coincide and the greater will be this common area. Hence a direct comparison of the common area with the total area of either curve gives a numerical expression of the efficiency of the distribution. If the distribution be perfect the two curves will coincide, and the ratio, common area to total area, will be unity. If the distribution is very poor the common area will be small in proportion to the total, and the ratio will be correspondingly low. Moreover, while it is not so evident from the present brief description of the method, a more careful analysis of the mutual relations of these two curves shows that two distribution curves, differing from one another in shape, but yielding the same ratio of common area to total area, represent conditions actually different, but identical as regards relative distribution.

If we define the term "excessive discharge" as that part of the discharge from a sprinkler which is flowing upon any area of the filter in excess of the mean rate for the total area (represented graphically by that portion of the area above the straight line), then we may define the coefficient of distribution as the ratio of the total discharge minus the excessive discharge to the total discharge; or, in algebraic form,

$$\text{Dist. Coef.} = \frac{T - E}{T};$$

or more simply,

$$\text{Dist. Coef.} = 1 - \frac{E}{T}.$$

Comparisons of the areas may be made quite readily by means of the planimeter. To obviate the necessity of multiplying to get the quantities, $Q \times D$, a special form of plotting paper has been used, which performs the multiplications graphically. This paper has one set of coördinate lines vertical, as usual, and the other set radiating from the origin. Direct plotting of the figures used in curve A upon such paper will in reality give curve B, any given value having an actual magnitude upon the plot which is proportional to the horizontal distance from the origin.

While the plot as thus worked out is essential to a clear understanding of the conditions and of the definition of the coefficient, it has been found much simpler and sufficiently accurate in practice to calculate the coefficient from the observed data (see table). Adding up the column $Q \times D$ gives the area under curve B, representing the total discharge (T). Adding the column D gives a number representing the total wetted area; that is, it bears the same relation to the total area as the sum of the Q × D column does to the total discharge. Dividing the former by the latter gives the mean discharge (M) per unit area for the whole filter. If this mean discharge be subtracted from each value of Q which is greater than the mean, a set of figures are obtained which are shown in the table under the heading Ex. Q. These figures are the excessive discharges per unit areas at the respective distances from the centre. Multiplied again by the distances in the D column, they give the excessive discharges for the concentric rings, and the sum of the latter figures, column D × Ex. Q, gives the total excessive discharge as already defined (E). We have now the two factors T and E for use in the formula,

$$\text{Dist. Coef.} = 1 - \frac{E}{T}.$$

In the illustration used,

$$E = 13.4,\ T = 73.1,\ c = 1 - 0.18 = 0.82.$$

The coefficient thus derived refers to the efficiency of the sprinkler under the given conditions and figured upon the area of the wetted circle. There remains to be derived the true coefficient based upon the total area of the filter, portions of which between the circles may not be wetted. Each sprinkler must then be referred to the area of a square whose side is the distance between centres of adjacent sprinklers. Denoting the area of the square by Sq and of the wetted circle by Cir, the final coefficient, C, may be found from the former coefficient, c, by the formula,

$$C = c \times \text{Cir}/\text{Sq}.$$

By the use of such a distribution coefficient it is possible to make experimental comparisons of both pressure and gravity sprinklers of various designs, and to test the effects of variations in head, rate of discharge, and other variables upon the distribution.

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CONTRIBUTION FROM THE SANITARY RESEARCH LABORATORY AND SEWAGE EXPERIMENT STATION OF THE MASSACHUSETTS INSTITUTE OF TECHNOLOGY

STUDIES OF SEWAGE DISTRIBUTORS FOR TRICKLING FILTERS

BY C.-E. A. WINSLOW, EARLE B. PHELPS, C. F. STORY, AND H. C. McRAE

I. THE GENERAL PROBLEM OF SEWAGE DISTRIBUTION

THE chief technical difficulty in the construction of the trickling, or percolating filter, as it is called in England, lies in securing efficient distribution of the sewage. The liquid must be discharged slowly and evenly over the whole surface of the bed and thoroughly aërated in the process; in other words, it must be distributed in a rain of fine drops. The mechanism adopted differs widely in various disposal plants. Some of the earlier English investigators used tipping buckets and troughs, placed at intervals over the filter, relying on the splash to dose the intervening areas. This method must necessarily produce imperfect results, and the required apparatus is cumbrous and expensive. Other devices are designed to secure a steady overflow from fixed level troughs or pipes. Of these, the most highly developed is the Stoddart distributor, in use at Horfield and other English towns. This is practically a series of channels over the sides of which the sewage overflows continuously, dripping from points on the under side, 360 points being allowed to a square yard. Theoretically this should secure a very even distribution; but the system requires that the channels remain perfectly level and clean, a condition difficult to preserve in practice.

Another series of distributors secures an even spread of the sewage by the use of some form of moving apparatus. The simplest devices of this type consist of a pair of revolving arms discharging sewage in opposite tangential directions, so that they are propelled by their own discharge on the principle of a Barker mill. The Candy-Whittaker patents cover some of the best known English distributors of this type. More elaborate designs involve the use of power for moving troughs from which the sewage trickles by numerous small openings, or from which it flows in a thin stream over a weir. In connection with the experiments carried out at Birmingham, Scott-Moncrieff installed a large distributor of this pattern, which consisted of a trough revolving on a circular rail at its outer extremity and driven by a small oil engine. At Hanley is a similar trough, which runs back and forth over a rectangular bed, propelled by an electric motor.

There are certain inherent objections to all such movable distributors which have so far seemed almost insuperable to American engineers. Power, or its equivalent hydraulic head, is necessarily involved in all of them. Movable arms and troughs involve structural details which are expensive to construct and which require constant attention; and it may seriously be doubted whether devices of such complexity and delicacy of adjustment could be operated at all under the influence of a severe winter climate. Even in England trouble was experienced at certain plants during the cold winter of 1906–07.

The simplest and most practical of the English methods of distribution is the system of spraying the sewage upward from sprinkler heads arranged at intervals along fixed pipes. Fixed sprinklers of this type have been used for some time at Salford and other English towns, and are planned for the two largest trickling filter plants yet designed in the United States — at Columbus, Ohio, and Waterbury, Connecticut.

The chance of offensive odors is of course increased by the distribution of sewage spray in the air, but the free oxidation probably facilitates purification. At Birmingham a careful comparison has been made of various types of revolving distributors and pressure nozzles. Mr. J. D. Watson reports (*Engineering News*, 58, 89), as a result of these studies, that spray jets cost $2,500 per acre to install, against $5,000 to $20,000 for moving distributors, and that the beds equipped with the latter were idle as a result of the distributor being out of order from 8 to 28 per cent. of the time. The results obtained with fixed sprinkler heads of course depend largely on the details of their construction, the head available, the spacing of the nozzles, and particularly on the construction of the nozzles themselves.

In the course of the investigations on the purification of the sewage of Boston, carried out at the Sanitary Research Laboratory of the

Massachusetts Institute of Technology, another method, that of gravity distribution, has been devised.

This system, referred to in an address before the Society of Arts, October 12, 1905, and described in the *Engineering News* for August 16, 1906, appears to possess certain distinct advantages. The sewage is distributed in troughs elevated over the filter, from which it drops through holes in the bottom of the troughs onto concave disks below, splashing upward from the disks in a fine spray. It has been used for two years with success at the Boston station, and has since been adopted by the Massachusetts State Board of Health for an experimental filter at Andover, according to an account published in the *Engineering News* of April 11, 1907.

With distributors of almost any type it is possible to combine siphon tanks or other automatic devices for securing intermittency of operation. The increased pressure thus produced at the moment of maximum discharge tends to keep clear any openings which might otherwise clog, and when the sewage is discharged under pressure from perforated pipes the variations in head tend to make the circle of spray both wider and more even. This plan has long been used at Chesterfield and other English towns.

II. METHODS FOR MEASURING THE EFFICIENCY OF SEWAGE DISTRIBUTORS

The Stoddart distributor and the various types of movable sprinklers used in England are theoretically perfect, and their failure, when it occurs, is due to structural imperfections or lack of care in maintenance. On the other hand, fixed sprinkler nozzles and the gravity system of distribution cover at best only a portion of the surface of the filter. Since both these latter types discharge sewage approximately in a circle, each nozzle or disk will wet only 78 per cent. of the square area surrounding it. Furthermore, almost all these distributors tend to discharge, for the most part, in one or two concentric rings of the wetted area. Some nozzles, too, discharge unevenly along different radii. It is important to determine how serious these deviations are. Imperfect distribution means that a part of the filter area below is wasted and another part is operated at excessive rates. Yet almost no exact data upon this point are now available. Some experimenters have noted the proportion of the total area wetted by their sprinklers; but such a standard is, of course, wholly inadequate, since it fails to discriminate the extent of the discharge upon different portions of the wetted area. We have therefore thought it worth while to make a somewhat extensive study of this point, comparing in detail the efficiency of various fixed sprinkler nozzles and of a number of patterns of gravity distributors under various conditions.

For the quantitative study of the efficiency of distribution a measuring apparatus was devised, the construction of which is indicated in Figures 1 and 2. It consisted of a shallow, circular basin of cement concrete 14 feet in diameter, sloping toward a central drain connected with a waste pipe below. A 2-inch supply pipe passed under the concrete and rose through this drain at the centre of the basin. To this pipe were attached the sprinkler nozzles of various types designed for pressure distribution. A wooden platform in the shape of the 30° sector of a circle was attached at its pointed end to a 2½-inch

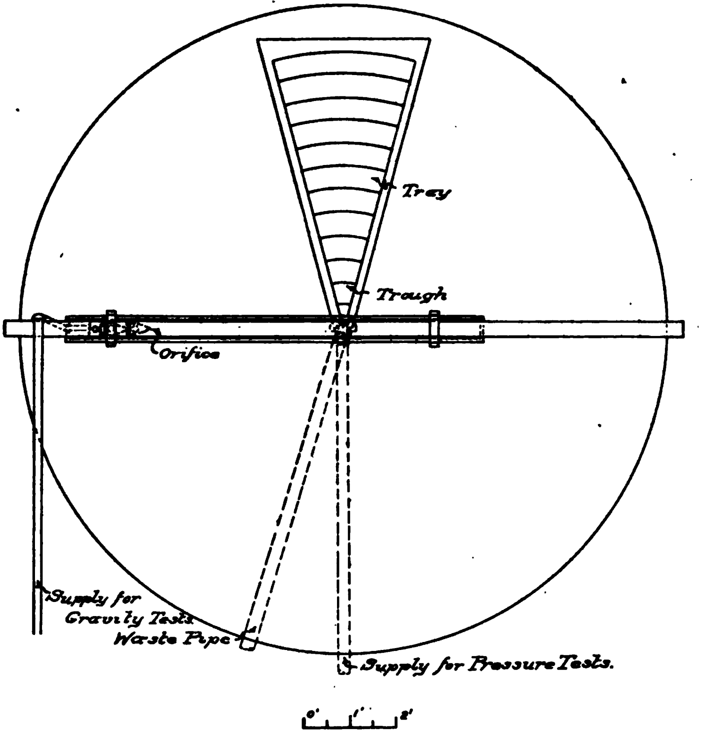

FIG. 1.—PLAN OF APPARATUS FOR SPRINKLER TESTS

iron collar, fitted over the supply pipe so as to turn freely upon it. The outer end of the wooden platform was supported on ball-bearing castors, arranged to run on a flat shelf at the periphery of the concrete basin, so that the whole platform could be revolved about the supply pipe as a centre; and the collar and castors were so adjusted that the platform

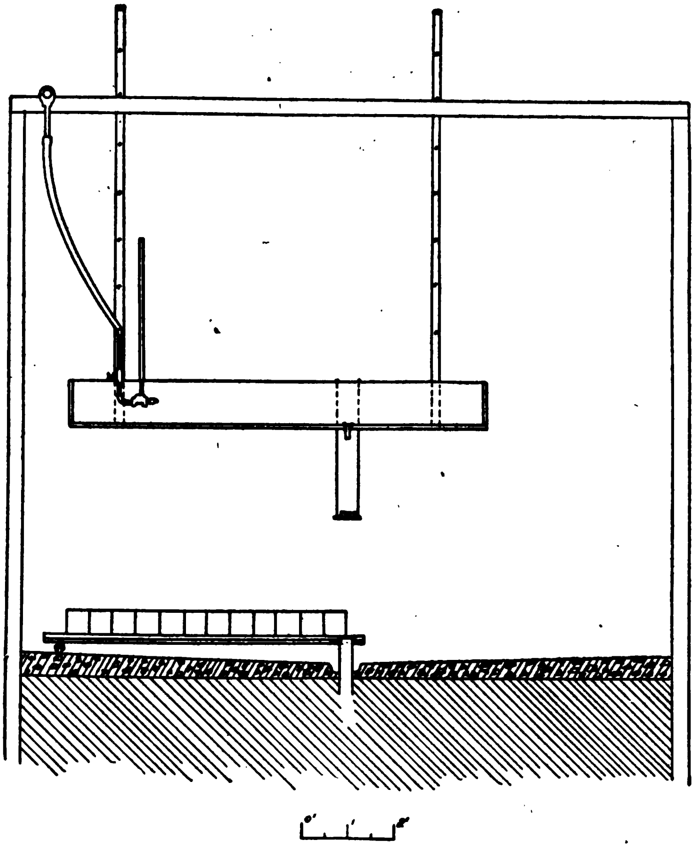

FIG. 2.—SECTIONAL ELEVATION OF APPARATUS FOR SPRINKLER TESTS

was exactly level. The platform carried a galvanized iron tank, also in the form of a 30° sector, divided into compartments by concentric arcs 6 inches apart. The radius of the tank was 6 feet, giving twelve concentric compartments, and its depth was 6 inches. When this tank was rotated about the supply pipe during the course of an experiment, the evenness of distribution was of course measured by the relative amount of liquid collected in the respective compartments. In various stationary positions the trough recorded radial inequalities.

The supply pipe at the centre of the basin was connected at its outer end with an upright stack of 2-inch pipe made up of short lengths, any desired number of which could be used. At the top of the stack was a wooden box, fed by a hose and relieved by an overflow just below the top. The difference between the water level in this box (varied by removing sections of pipe from the upright stack) and the level of the sprinkler nozzle at the centre of the basin represented approximately the total head effective in distribution. The rate of flow was of course fixed by the head and the sprinkler nozzle used. It was estimated in each test from the total discharge into the measuring tank, and from the time occupied by the run.

The apparatus as shown in Figures 1 and 2 was adapted for the study of gravity distributors by the addition of an overhead trough. This was hung from two timbers passing diametrically across the basin, supported at their extremities by upright studs. At a point fixed over the centre of the apparatus a ¾-inch brass orifice in the bottom of the trough allowed the liquid to discharge upon the splashing disk below.

The uprights supporting the trough were fixed to the cross-timbers above by movable iron pins and the uprights were pierced for the reception of these pins at intervals of 1 foot, so that the distance between the trough and the measuring tank below could be varied at will. The distance between the level of the water in the trough and the top of the tank measured the total head available for gravity distribution. In this system, however, there is another variable. With a given total head, the distance between the point of discharge and the splashing disk and the distance from the splashing disk to the filter may be mutually varied. In our experiments this was provided for by supporting the various splashing disks tested upon a circular lead plate, 6.5 inches in diameter, hung from the distributing trough by four wires. Varying the length of these wires altered the portion of the total head above the splashing disk (called for convenience "head-on-sprinkler").

The rate of flow with this type of distributor is, within limits, independent of the head, so that the rate also had to be taken into account. The rate of discharge was equal to the rate of supply of liquid to the trough, and this supply was measured through a calibrated orifice, indicated in Figures 1 and 2. Two orifices were used in the experiments, one $\frac{7}{16}$ inch and one ⅝ inch in diameter. The orifice plug in use was connected to a 2-inch tee by a reducing coupling. The section of the tee was large enough to permit an accurate measurement of the static head on the orifice by a glass piezometer tube inserted in its top, and the head was adjusted at any desired point by a valve behind the tee.

The orifices were calibrated by measuring their discharge at various heads in a cypress tank, 2 feet by 6 feet in area and 3 feet deep. A special form of gauge, to be described later, was used in these measurements, and they were accurate within .005 inch in the depth of the tank, or .005 cubic foot of discharge. The calibration of the $\frac{1}{16}$-inch orifice is shown in Table I.

TABLE I—CALIBRATION OF ORIFICE

Head on $\frac{1}{16}$-inch orifice.	Q in cubic feet per minute.	Q in gallons per minute.
Inches.		
5	.228	1.71
6	.240	1.80
7	.256	1.91
8	.276	2.06
9	.289	2.16
10	.306	2.29
12	.330	2.47
14	.354	2.65
16	.378	2.83
18	.400	3.00
20	.420	3.15
22	.438	3.28
25	.468	3.50
36	.546	4.08

The discharges of 1.71 and 4.08 gallons per minute, corresponding to 5-inch and 36-inch heads on the orifice, were chosen as two arbitrary rates of discharge for the gravity distributor tests. The ⅝-inch orifice was tested sufficiently to find a third, higher rate. A head of 18 inches on this orifice was found to give a discharge of 1.1325 cubic feet, or 8.47 gallons per minute, and this was used as the standard rate for high discharges. The entire apparatus was protected during the course of the experiments by a rough wind shield.

The measurement of the volume of liquid collected in the various compartments of the measuring tank during a test was, according to our original plans, to be made directly, and for this purpose a small stopcock was attached to each compartment. This process proved somewhat tedious, however, and the same end was attained by measuring the depth of the liquid in each compartment at the end of the run, after a careful preliminary calibration of the various compartments.

The depth of water in this measuring tank, and in the cypress tank used for nozzle calibration, was measured by a simple and accurate gauge devised for the purpose, the construction of which is indicated in Figure 3. It consisted of a graduated glass tube (a 1 c.c. pipette), the graduated portion being 6 inches long and divided into hundredths. This tube slid up and down in a cork fitted into a long, flat piece of wood, carefully planed. When the wooden crosspiece rested in a horizontal position, supported at its ends, the tube between was exactly vertical. Just above the wooden crosspiece a fine wire was stretched longitudinally between two supports behind the tube, and behind the wire a small mirror was fixed. By reading with the wire and its reflection superposed the position of the tube could be estimated to tenths of its smallest graduations (.0005 foot). In measur-

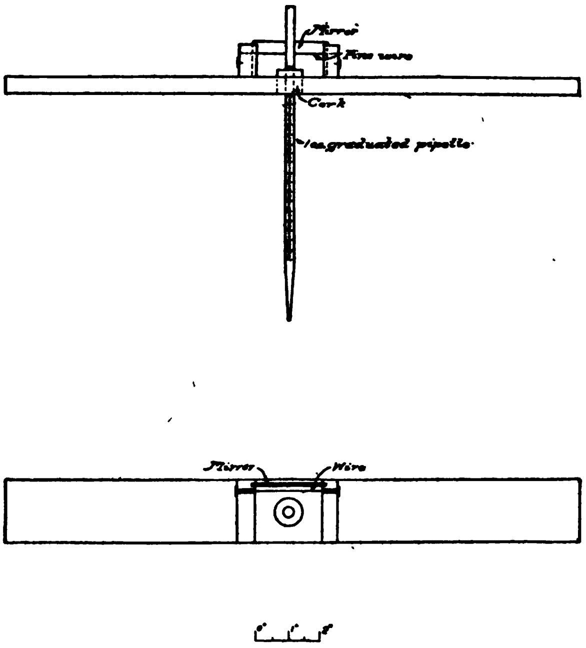

Fig. 3.—ELEVATION AND PLAN OF GAUGE

ing the depth of water in a compartment of the measuring tank the wooden crosspiece of the gauge was rested on the two sides of the tank and the tube was slid carefully down till its mouth just touched the surface of the water. This point could be detected with great accuracy by the jump into the tube of the drop of water drawn up by capillary action.

Each compartment of the measuring tank was calibrated by pouring in measured quantities of water and reading the depth after the addition of each successive portion. The average of these determinations gave the number of units of depth corresponding to a given volume of water. The reciprocal of this was the volume of water corresponding to a unit of depth for the compartment in question. A special correction was necessary for irregularities in the bottom of the tank and for the zero reading of the gauge. The former was determined by the difference between the change in level produced by the first portion of water poured in and by subsequent portions. The general results of this tank calibration are indicated in Table II.

TABLE II — CALIBRATION OF MEASURING TANK

Compartment.	Change in depth for one liter added.*	Bottom correction.	Liters per unit change in depth.†	Gallons per unit change in depth.	Relative value (area).
1	.268	+.020	.933	.246	1.0
2	.187	. .	2.674	.706	2.87
3	.220	—.017	4.545	1.209	4.92
4	.158	—.009	6.329	1.67	6.80
5	.120	—.008	8.333	2.20	8.96
6	.101	—.017	9.901	2.61	10.6
7	.084	—.017	11.90	3.14	12.8
8	.074	. .	13.51	3.57	14.6
9	.065	—.015	15.38	4.06	16.6
10	.059	—.014	16.95	4.47	18.2
11	.052	—.027	19.23	5.08	20.7
12	.048	—.013	20.83	5.51	22.4

* Half a liter for compartment No. 2; quarter of a liter for compartment No. 1.
† Unit = .5 foot.

The "relative values" in the last column of the table should, of course, have come out 1, 3, 5, 7, etc., if there had been no irregularities in the construction of the tank, for with unit depth:

$$Q_1 = \frac{\pi \times 1^2}{4} \times 1 = \frac{\pi}{4}.$$

$$Q_2 = \frac{\pi \times 2^2 \times 1}{4} - \left(\frac{\pi \times 1^2}{4} \times 1\right) = \frac{3\pi}{4} = 3Q_1.$$

$$Q_3 = \frac{\pi \times 3^2 \times 1}{4} - \left(\frac{\pi \times 2^2}{4} \times 1\right) = \frac{5\pi}{4} = 5Q_1.$$

The testing apparatus in operation is shown in Figure 4, and nearer views of the measuring tank in Figures 5 and 6. The following memorandum of the routine procedure in making the test of a gravity distributor may make the working of the whole apparatus somewhat clearer. In beginning a run the measuring tank was emptied, put in place on the wooden platform, and covered with a cover of tar paper stretched over slats. The desired total head was adjusted by raising or lowering the trough, and the head on the sprinkler was regulated by means of the wires suspending the lead plate from the trough. The disk to be tested was then centred on the lead plate just below the ¾-inch opening in the bottom of the trough. The valve supplying the trough was then opened and regulated till the desired discharge was registered on the

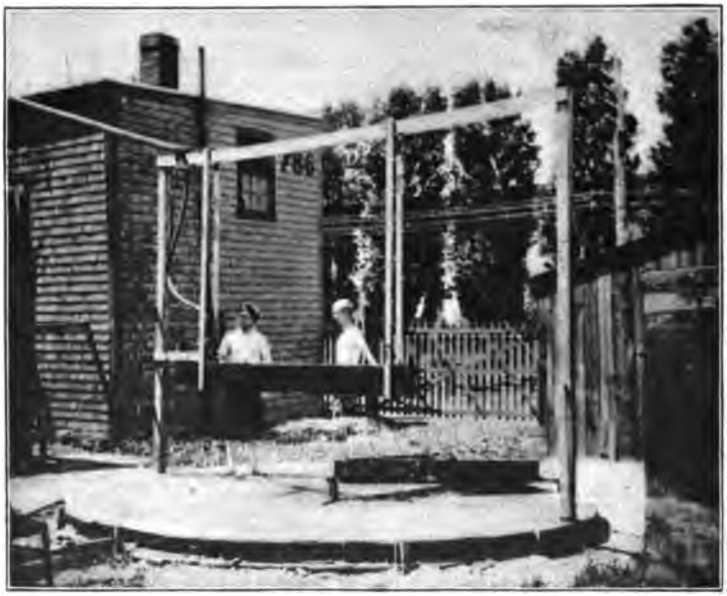

FIG. 4.—GENERAL VIEW OF TESTING APPARATUS WITH TEST IN PROGRESS

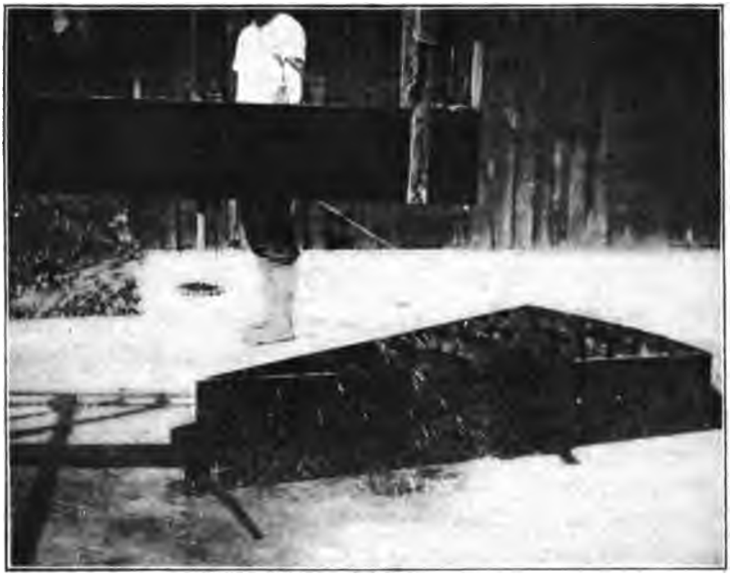

FIG. 5.—SIDE VIEW OF MEASURING TRAY WITH TEST IN PROGRESS, SHOWING TROUGH AND DISTRIBUTING DISK IN PLACE

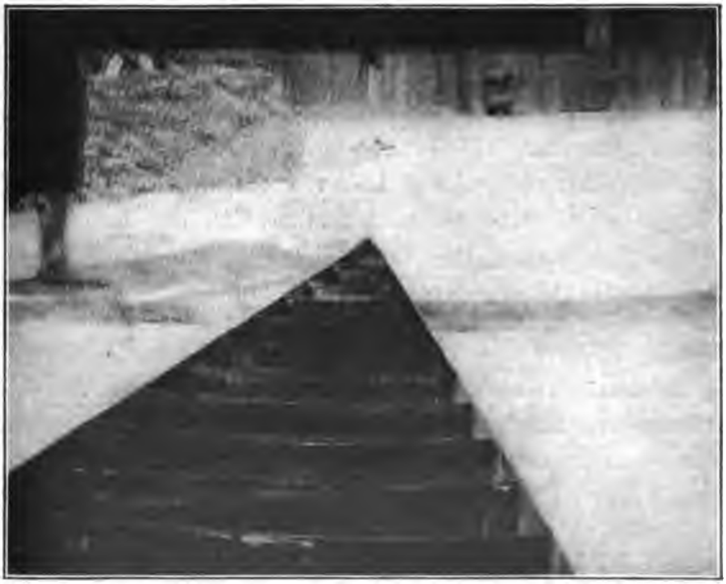

FIG. 6.—VIEW OF MEASURING TRAY FROM ABOVE. TEST IN PROGRESS

piezometer tube. After conditions had become constant in the trough, the cover of the measuring tank was removed (the time being taken with a stop watch). During the test the tank was slowly rotated by hand. As soon as any compartments became so full as to threaten splashing the test was stopped, the cover replaced, and the time again noted. The supply valve was then closed, and after the discharge had ceased the cover was removed and the depth of liquid in each compartment of the measuring tank was determined, as described above.

III. METHOD OF CALCULATING A COEFFICIENT FOR THE EFFICIENCY OF SEWAGE DISTRIBUTORS

After the proportionate amount of discharge in various zones about the distributors had been determined by the method outlined above, it was necessary to devise some method of stating the results so that the efficiency of the various types could be easily and accurately compared. For this purpose one of us (E. B. P.) has derived a "coefficient of distribution," which shows the mathematical relation between any observed condition of distribution and that ideal condition in which each small unit of area should receive its exact proportionate share. Professor Phelps' calculations were published in the *Engineering News* of October 11, 1906, and in the TECHNOLOGY QUARTERLY for March, 1907 (20, 34). The following quotation outlines his method:

"For the purposes of this problem let us assume that a pressure sprinkler is shooting up a spray of sewage fountain-wise, the sewage falling over a circular area. Along any radius let us place a line of small, square collecting tanks of known areas, which we may designate the unit area. Let it be assumed, further, that the distribution along this radius is the mean of that along all radii. The variations along the various radii will be small, and the assumption may be completely realized, as is being done in our experiments, by revolving the line of collecting tanks about the centre during the experiment. After running a suitable length of time, the sewage is shut off, and the amount collected in each small tank is measured. In the accompanying table a set of actual results is reproduced for the sake of illustration.

TABLE III—RESULTS OF OBSERVATIONS OF DISTRIBUTION EFFECTED BY SEWAGE SPRINKLER

Tank No.	Radial distance (D).	Sewage quantity (Q).	D × Q.	Excessive quantities (Ex. Q).	D × Ex. Q.
1	0.5	0.8	0.4	. .	. .
2	1.5	1.6	2.4	0.14	0.21
3	2.5	2.7	6.8	1.24	3.10
4	3.5	2.5	8.8	1.04	3.65
5	4.5	2.3	10.3	0.84	3.78
6	5.5	1.9	10.4	0.44	2.43
7	6.5	1.5	10.0	0.04	0.26
8	7.5	1.3	9.8	. .	. .
9	8.5	1.0	8.5	. .	. .
10	9.5	0.6	5.7	. .	. .
Totals	50.0		73.1		13.43

$73.1 \div 50.0 = 1.46 = M.$

"In the first column are given the numbers of the collecting tanks, No. 1 being at the centre. In column 2, headed D, are given the distances from the centre of the sprinkler to the centres of the corresponding tanks, the side of the tank being taken as unit distance. In the third column, Q, are given the amounts of sewage measured in each tank at the end of the test. Any unit of volume may be employed. In this case the depths in the measuring tanks are used, these being proportional to the volumes. Now if these quantities, Q, are plotted as ordinates against the corresponding D values as abscissæ, we obtain a curve showing the relative distribution of the sewage along the radius. This curve is shown at A in the diagram (Fig. 7). It shows the rate of discharge per unit area at any point whose distance from the centre is known, and will be known as the curve of radial distribution. Since the radial distribution measured was assumed to be the mean of all radial distributions, the ordinate of this curve at any given distance from the centre shows the rate of discharge at all points on a circumference described about the centre at the given distance from it. Hence to obtain a curve showing the distribution on the whole circle, *i. e.*, on an infinite series of successive circumferences, it is necessary to multiply the ordinate for each point on the radius by the length of the corresponding circumference, or what amounts to the same thing, relatively, by the radial distance of the point in question. In column 4, D × Q, in the table are given the figures resulting from this operation. These quantities represent the relative rates of discharge of the sewage upon successive concentric rings of the circle, the width of the rings being unity and their distances from the centre being the corresponding D. If now a new curve be plotted with these products just obtained as ordinates and the corresponding D values as abscissæ, the resulting curve is the curve of distribution over the wetted area (curve B in the figure).

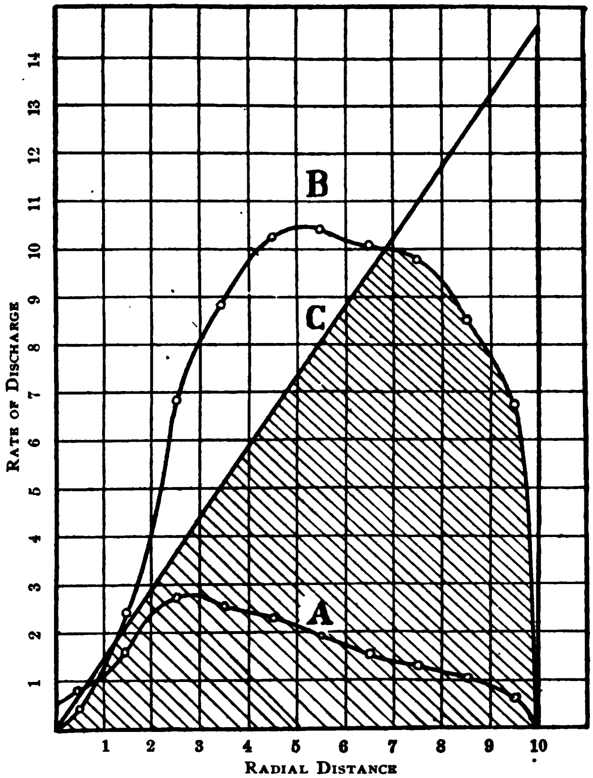

FIG. 7.—DIAGRAM OF DISTRIBUTION

A. Curve of radial distribution. B. Curve of distribution over the circle. C. Curve of perfect distribution over the circle.

" From its construction this curve has the following properties: The ordinate at any point shows the relative rate of discharge of the sewage upon a circumference at the corresponding distance from the centre;

the area of any vertical strip shows the relative total discharge upon the corresponding ring of the wetted circle; and the total area under the curve represents the total discharge from the sprinkler.

"It will next be necessary to construct the curve of perfect distribution. This curve must conform to the fact that the discharge upon all unit areas is the same, and hence that discharges upon all circumferences are directly proportional to their lengths, *i.e.*, to their radii. Obviously, then, the curve is a straight line passing through the origin. Its slope is determined graphically by the fact that the area under the curve represents the total discharge from the sprinkler, and hence equals the area under the distribution curve already drawn. The latter area may be determined by means of a planimeter, and from the known base of the triangle its altitude may be computed. It may be calculated more readily but less accurately by adding up the column Q × D, and dividing by the radius of the wetted circle (10). This curve is shown in the figure as curve C.

"We now have the curve of actual distribution and the curve of perfect distribution. There remains to be derived a mathematical expression of the relation between these two curves. The curves have a common area which is shaded in the diagram. From what has preceded it will be apparent that the more perfect the distribution the more nearly will the two curves coincide, and the greater will be this common area. Hence a direct comparison of the common area with the total area of either curve gives a numerical expression of the efficiency of the distribution. If the distribution be perfect the two curves will coincide, and the ratio, common area to total area, will be unity. If the distribution is very poor the common area will be small in proportion to the total, and the ratio will be correspondingly low. Moreover, while it is not so evident from the present brief description of the method, a more careful analysis of the mutual relations of these two curves shows that two distribution curves differing from one another in shape, but yielding the same ratio of common area to total area, represent conditions actually different, but identical as regards relative distribution.

"If we define the term 'excessive discharge' as that part of the discharge from a sprinkler which is flowing upon any area of the filter in excess of the mean rate for the total area (represented graphically by that portion of the area above the straight line), then we may define the coefficient of distribution as the ratio of the total discharge minus the excessive discharge to the total discharge; or, in algebraic form.

$$\text{Dist. Coef.} = \frac{T - E}{T};$$

or, more simply,

$$\text{Dist. Coef.} = 1 - \frac{E}{T}.$$

"Comparisons of the areas may be made quite readily by means of the planimeter. To obviate the necessity of multiplying to get the quantities Q × D, a special form of plotting paper has been used, which performs the multiplications graphically. This paper has one set of coördinate lines vertical, as usual, and the other set radiating from the origin. Direct plotting of the figures used in curve A upon such paper will in reality give curve B, any given value having an actual magnitude upon the plot which is proportional to the horizontal distance from the origin."

While the plot and the foregoing demonstration are essential to a clear understanding of the coefficient, it has been found much simpler and sufficiently accurate in practice to make an arithmetical calculation of the coefficient directly from the observed data. Either of the last two columns in Table II may be used instead of column 2, D, in Table III, since it corresponds to it with an allowance for the irregularities of construction in the tank. For column 3, Q, the observed depth is used, after the application of the proper bottom correction. The sum of the fourth column, D × Q, gives the total discharge, T, upon the sector tested, directly in gallons, if the relative value in gallons is used for D. The sum of column 2, D, corresponds to the total wetted area; that is, it bears the same relation to the total area that the sum of the fourth column, D × Q, does to the total discharge. In practice, when the liquid discharged did not reach the outer compartments, those zones not wetted were of course omitted from column 2. Now if the sum of column 4, D × Q, be divided by the sum of column 2, D, the quotient, M, will be the mean discharge per unit area over the whole wetted surface. If this mean discharge be subtracted from each value of Q which is greater than the mean, a set of figures are obtained which are shown in Table III in column 5, under the heading Ex. Q. These figures are the excessive discharges per unit areas at the respective distances from the centre. Multiplied again by the distances in the D column they give the excessive discharges for the concentric rings, and the sum of the latter figures, column D × Ex. Q, gives the total excessive discharge as already defined, E. In practice, the difference between the mean discharge and the actual discharge was also determined for the zones which received less than their share; and the sum of these values was used as a check on the total excessive discharge, E.

Either the total excessive discharge on the areas receiving too much liquid, or the total deficient discharge on the areas receiving too little, when divided by the total discharge measures the deviation of the distribution from a perfect condition. Referring to Figure 7, it will be seen that the total discharge, T, corresponds to the area under the line C, and that the excessive discharge, E, corresponds to the area below B and above C. Perfect distribution would be indicated by a value of zero for E or a coincidence of lines B and C. This condition is called unity, and the observed condition in any case is expressed as the difference between unity and the ratio of excessive discharge to total discharge. In the shape of a formula this becomes,

$$\text{Dist. Coef.} = 1 - \frac{E}{T},$$

and in the illustrative example given in Table III,

$$E = 13.4,\ T = 73.1,\ c = 1 - 0.18 = 0.82.$$

The crude coefficient thus derived refers to the efficiency of the sprinkler under the given conditions and figured upon the area of the wetted circle. There remains to be derived the true corrected coefficient based upon the total area of the filter, including the unwetted corners between the circles. Each wetted area should be referred to the area of a square whose side is the distance between centres of adjacent sprinklers. Denoting the area of the square by Sq and of the wetted circle by Cir, the corrected coefficient, C, may be found from the crude coefficient, c, by the formula,

$$C = c \times \text{Cir}/\text{Sq}.$$

This correction will, of course, diminish the crude coefficients to less than 78 per cent. of their value for the wetted area, and all coefficients recorded in our investigation are calculated upon this basis of arrangement in squares. In practice, however, it would be of advantage to arrange distributors, not along two axes at right angles to each other, so that each is at the centre of a square, but on three axes inclined at 120°, so that each is at the centre of a hexagon. With distributors at the centres of squares, the waste area, with no overlapping, amounts to 21.5 per cent. of the whole. With distributors at the centres of hexagons, the waste area amounts to only 9.9 per cent. The hexagonal arrangement makes it possible to put 343 sprinkler heads on an acre instead of 303, spaced with 12 feet between centres, and to raise a rate of 2,000,000 gallons to 2,270,000 without increasing the discharge upon any portion of the wetted area.

IV. Determination of Coefficients for Gravity Distributors

The system of gravity distribution involves the discharge of the sewage from openings in the bottom of troughs onto concave disks placed below, the discharged stream having sufficient force to splash up and out from the disks in a fine spray.

The splashing disks first used on the Technology trickling filter in 1905 were of metal, 3 inches in diameter, and with a concavity corresponding to a radius of curvature of 6 inches. These were at first placed only 5 feet apart, with a discharge on each disk of less than .9 gallon per minute. This discharge proved too low for good distribution, and a single disk was substituted for four on each 100 square feet in area, giving 10 feet between disks and a discharge on each of 3.47 gallons per minute. This proved far more satisfactory, and the experimental filter thus equipped gave very satisfactory results even through the severe winter of 1906-07. The performance of one of these disks in operation is shown in Figure 8.

With gravity distributors there are three distinct variable factors, the rate of discharge, the distance between the trough and the disk, and the distance between the disk and the filter below. In our experiments we have varied all three independently. For rates we have used the three standard discharges described above, corresponding, respectively, to 1.71, 4.08, and 8.47 gallons per minute. For the total head between the trough and the measuring tank (corresponding to the filter surface) three standard heads were used, 2 feet, 4 feet, and 6 feet; and with each of these total heads three different positions of the disk were tested. With the 2-foot total head the head-on-sprinkler was made 6 inches, 12 inches, and 18 inches; with the 4-foot total head the head-on-sprinkler was made 1, 2, and 3 feet; with the 6-foot total head the disk was placed at 1, 2½, and 4 feet.

Besides these three variable factors the efficiency of gravity distribution depends upon the design of the splashing disks themselves.

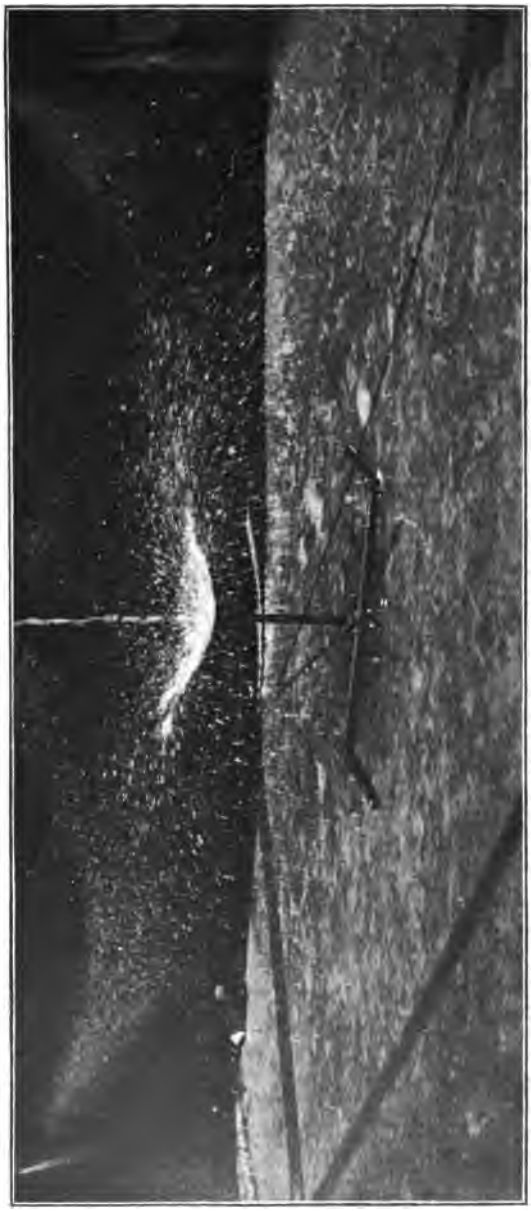

FIG. 8.—VIEW OF GRAVITY DISTRIBUTOR IN OPERATION

Twenty-three types of disks were tested in our experiments, varying in size and curvature, in the hope of finding some more favorable design than the simple concave type first used. Cross-sections of the disks examined, with their respective dimensions, are shown in Figure 9.

Nos. 1 to 4 were wooden disks 2 inches in diameter, with surfaces of different degrees of concavity varying from a radius of curvature of 4 inches to one of 20 inches. Nos. 5 to 10 were disks 3 inches in diameter, Nos. 6 and 7 of metal and the rest of wood, the radii of curvature of their concave surfaces varying from 2 to 20 inches. Nos. 11, 12, and 16 were larger, very shallow wooden disks. Nos. 13, 14, 17, 18, and 19 were wooden disks whose upper surfaces were concave about a central raised point, rising again all around their outer edge. We have called this design the mushroom disk. No. 15 was a large, deep metal disk, 5 inches in diameter, and with a 3-inch radius of curvature. Finally, Nos. 20, 21, 22, and 23 were large, 6-inch wooden disks with radii of curvature varying from 4 to 20 inches.

Since each of these twenty-three disks was tested under twenty-seven standard conditions (three subdivisions of each of three total heads at three rates), making over six hundred tests in all, it has not seemed best to publish the mass of figures obtained in detail. The final corrected coefficients are given in Tables IV, V, and VI, arranged first according to the rate of discharge on each disk and then according to the head.

It must be remembered, of course, that the corrected coefficients in these tables take account both of the extent and the evenness of the circle of spray. Some of the disks produced good distribution within a small circle, but the rate on the wetted area was so great as to be excessive. By reducing the crude coefficients for such disks so as to bring the total rate down to 2,000,000 gallons per acre per day, the corrected coefficients have been estimated. This was done for the three rates of discharge tested as follows: A discharge of 1.71 gallons per minute on each splashing disk corresponds to 2,480 gallons per day per disk. At a 2,000,000-rate this requires 808 sprinklers per acre, or one sprinkler to each 54 square feet of filter area; the distance between sprinklers must therefore be 7.35 feet. Similarly the 4.08-gallon rate would require 341 sprinklers per acre, or one to each 128 square feet, and the distance between sprinklers must be 11.3 feet. The 8.47-gallon rate calls for 164 sprinklers per acre, or one to 266 square feet, with a distance of 16.3 feet between sprinklers. All coefficients in the

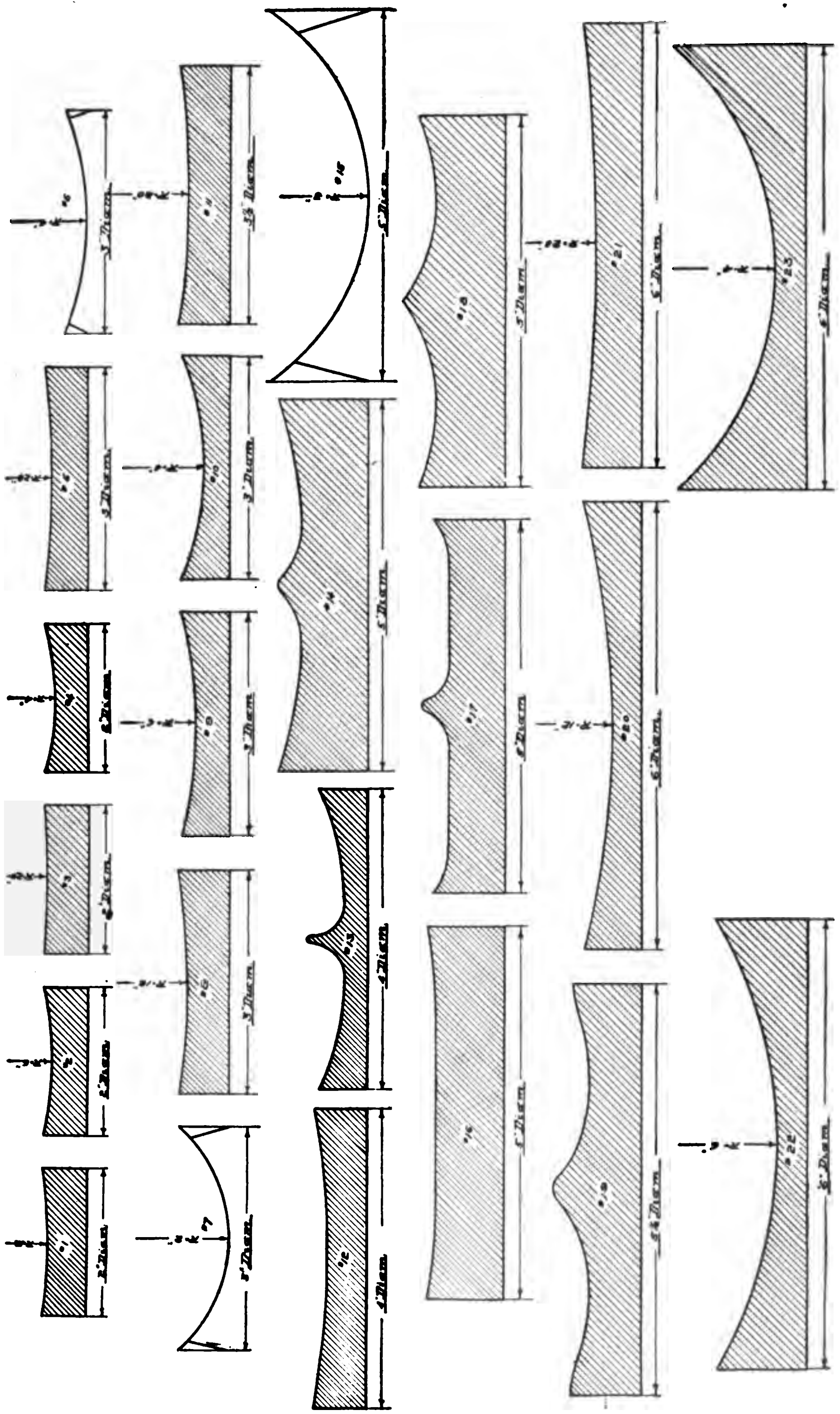

FIG. 9.—CROSS-SECTIONS OF SPLASHING DISKS

TABLE IV — CORRECTED COEFFICIENTS FOR GRAVITY DISTRIBUTORS

Rate on disk = 1.71 gallons per minute. Rate on filter = 2,000,000 gallons per acre per day

Total head in feet.	Head-on-sprinkler, feet.	Number of Disk.																						
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
2	0.5	.05	.07	.05	.05	.01	.02	.05	.01	.04	.05	. .	.02	.02	. .	. .	.02	. . .	. .	. .	. .	. .	. .	. .
	1.0	.10	.11	.06	.13	.04	.11	.11	.10	.04	.12	.04	.04	.05	.05	. .	.04	. .	.05	. .	.01	.04	. .	. .
	1.5	.06	.05	.10	.12	.05	.10	.09	.09	.05	.09	.05	.04	.05	.06	. .	.03	.06	.06	.02	. .	.02	. .	. .
4	1.0	.28	.28	.30	.30	.18	.21	.22	.17	.19	.31	.15	.17	.17	.16	. .	.15	.11	.10	.05	.08	.09	. .	. .
	2.0	.19	.29	.29	.37	.12	.27	.25	.16	.17	.40	.08	.10	.17	.17	. .	.08	.10	.17	.05	.05	.04	. .	. .
	3.0	.18	.14	.27	.27	.16	.30	.30	.17	.17	.23	.15	.15	.16	.15	. .	.08	.13	.15	.10	.01	.01	.01	. .
6	1.0	.29	.41	.43	.42	.08	.30	.30	.16	.17	.40	.14	.16	.27	.20	. .	.10	.17	.21	.09	.04	.05	. .	. .
	2.5	.41	.43	.43	.55	.38	.37	.38	.25	.26	.55	.33	.34	.37	.29	. .	.26	.31	.38	.17	.09	.04	. .	. .
	4.0	.41	.41	.41	.55	.35	.50	.35	.23	.26	.46	.16	.18	.40	.28	. .	.14	.23	.37	.16	.07	.10	.12	. .

TABLE V — CORRECTED COEFFICIENTS FOR GRAVITY DISTRIBUTORS

Rate on disk = 4.08 gallons per minute. Rate on filter = 2,000,000 gallons per acre per day

Total head in feet.	Head-on-sprinkler, feet.	NUMBER OF DISK.																						
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
2	0.5	.05	.05	.04	.05	.05	.05	.05	.04	.05	.05	.04	.05	.05	.03	. .	.05	. .	. .	.02	.08	.03	.02	. .
	1.0	.08	.07	.07	.08	.05	.08	.11	.08	.08	.09	.04	.06	.09	.10	.04	.09	.08	.09	.05	.06	.05	.06	. .
	1.5	.08	.08	.05	.07	.05	.11	.09	.08	.08	. .	.05	.08	.09	.09	.06	.09	.09	.09	.05	.06	.08	.08	.04
4	1.0	.20	.17	.17	.18	.15	.22	.20	.17	.17	.22	.15	.19	.18	.16	.09	.18	.18	.15	.07	.16	.13	.13	. .
	2.0	.24	.25	.23	.31	.18	.35	.32	.19	.26	.30	.17	.21	.28	.27	.09	.20	.20	.27	.10	.15	.09	.18	.10
	3.0	.19	.17	.18	.25	.16	.26	.40	.18	.18	.31	.17	.20	.26	.31	.14	.18	.27	.20	.13	.12	.11	.17	.09
6	1.0	.30	.29	.25	.30	.24	.26	.24	.25	.26	.24	.23	.26	.27	.25	. .	.25	.26	.29	.25	.19	.17	.18	. .
	2.5	.26	.46	.38	.51	.38	.50	.60	.41	.40	.42	.33	.36	.57	.54	.20	.21	.35	.37	.27	.27	.22	.18	.11
	4.0	.36	.38	.28	.44	.31	.56	.62	.36	.42	.44	.26	.32	.46	.45	.25	.30	.30	.33	.35	.25	.18	.24	.14

TABLE VI — CORRECTED COEFFICIENTS FOR GRAVITY DISTRIBUTORS

Rate on disk = 8.47 gallons per minute. Rate on filter = 2,000,000 gallons per acre per day

Total head in feet.	Head-on-sprinkler, feet.	NUMBER OF DISK.																						
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
2	0.5	.03	.03	.03	.05	.03	.05	.06	.04	.05	.05	.03	.04	.03	.04	.02	.04	.04	.04	.04	.04	.04	.03	.03
	1.0	.05	.05	.05	.07	.03	.07	.09	.05	.05	.07	.04	.05	.07	.07	.05	.07	.07	.06	.04	.05	.04	.05	.04
	1.5	.03	.06	.04	.05	.03	.09	.10	.05	.05	.08	.04	.04	.10	.10	.07	.07	.10	.08	.04	.05	.05	.06	.06
4	1.0	.10	.12	.11	.14	.09	.15	.14	.13	.14	.15	.12	.15	.15	.16	.09	.14	.16	.17	.09	.12	.11	.10	.09
	2.0	.12	.12	.11	.15	.09	.16	.18	.11	.13	. .	.10	.12	.20	.19	.11	.15	.18	.19	.12	.12	.10	.13	.09
	3.0	.11	.11	.10	.16	.09	.23	.24	.08	.09	. .	.10	.13	.23	.24	.13	.13	.20	.16	.09	.12	.10	.13	.09
6	1.0	.15	.15	.16	.17	.14	.18	.18	.16	.16	. .	.13	.17	.20	.19	.12	.15	.18	.18	.14	.15	.13	.13	.10
	2.5	.23	.23	.20	.23	.16	.24	.25	.21	.20	. .	.17	.20	.28	.30	.17	.20	.28	.30	.25	.20	.20	.22	.16
	4.0	. .	. .	. .	. .	.19	. .	.28	.18	.18	. .	.18	.22	.32	. .	.19	.30	. .	. .	. .	.18	.18	.18	.14

tables have therefore been corrected by referring the actual discharges to squares respectively 7.35, 11.3, and 16.3 feet on a side.

Taking up the independent variables tested in their order it is at once apparent that the efficiency of gravity distributors is markedly influenced by the rate of discharge on each disk, and that about 4 gallons a minute is the most favorable rate. The lower rate, 1.7 gallons per minute, gave very poor results with the larger disks, the liquid merely dripping over the edge. With the smaller disks fairly good results were obtained within a radius of 3 or 4 feet of the disk, although there was generally an excess of discharge nearest the centre. As the distance between sprinklers for this head was only 7.35 feet, the crude coefficients of .6 and .7 were not so greatly reduced as in other cases, and for the smallest (2-inch) disks this rate is better than either of the higher ones. Most of the 2-inch disks gave corrected coefficients of over .4 even with the 6-foot total head, and coefficients of over .5 were obtained for disks 4, 6, and 10.

The intermediate rate of 4.08 gallons per disk per minute did not suit the small, 2-inch disks as well as the lower rate, but with larger disks it gave better results. With good heads the liquid was well distributed, showing neither the concentration near the disk obtained with the lowest rate nor the marked outer ring characteristic of the higher one. Crude coefficients of over .7 were obtained with almost all the disks by using a 6-foot head, and the better ones ranged between .8 and .9. Furthermore, the wetted area was proportionately larger than with the higher or the lower rate of discharge; 4.08 gallons per disk per minute calls for a distance of 11.3 feet between sprinklers, and with high heads the better disks wetted circles 10 and 11 feet in diameter. The corrected coefficients remained, therefore, relatively high, several of the disks showing values over .5, and one, disk No. 7, over .6.

With the highest rate the better disks covered a 10-foot circle, but there was usually a ring of excessive discharge at a distance of 3 or 4 feet from the centre. The larger disks, at high heads, gave crude coefficients of between .7 and .8, wetting a circle of 6 feet radius. Corrected for a 16.3-foot square these crude coefficients are seriously reduced, only two of the corrected coefficients in Table VI being over .3.

The head used with the sprinkler disks was a most important factor in controlling their efficiency. The 2-foot total head proved inadequate in most cases. Some of the crude coefficients were fairly good (between .6 and .8); but the wetted area was always much too

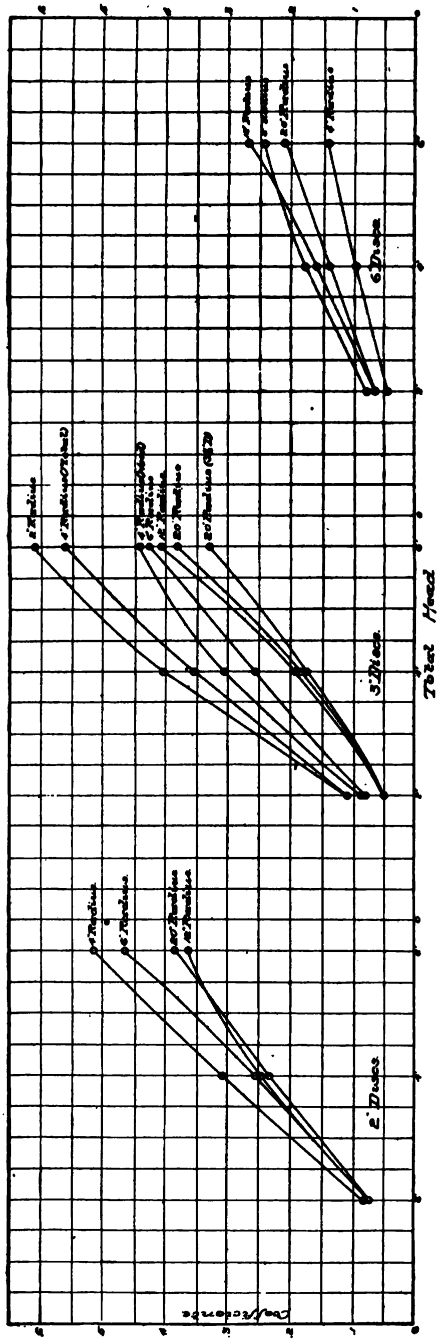

FIG. 10.—RELATION BETWEEN TOTAL HEADS AND COEFFICIENTS FOR DISKS OF DIFFERENT RADII OF CURVATURE Rate = 4.08 gallons per minute. Best head-on-sprinkler in each case.

small, giving an excessive rate near the sprinkler, and the corrected coefficients were almost always under .1. The 4-foot total head showed better results, and the 6-foot better still, the improved efficiency being directly proportional to the increase in head. This relation is indicated graphically in Figure 10 for the disks of 2, 3, and 6-inch diameters at the best rate of 4.08 gallons per disk per minute. The total heads are plotted as abscissæ and the best coefficients at the different total heads as ordinates. Each curve represents the results for one sprinkler disk, and the fifteen curves are arranged in three groups according to their diameters. The radius of curvature of each disk is indicated on the plot.

With regard to the best proportion of the total head to be used in the discharge above the disk, results are somewhat more variable. Increasing the head on the disk gives an enlargement of the wetted area, but it apparently brings about also an increased tendency to concentrated discharge in a single zone. The figures for the intermediate rate and 6-foot total head at the bottom of Table V include the best coefficients, and are fairly typical of the other results. They show that with a 6-foot total head a head-on-sprinkler of 2½ feet is most favorable for the shallower sprinklers, and a value of 4 feet is better for the deeper ones. The 1-foot head-on-sprinkler is always inadequate to secure a proper splashing discharge. These results are indicated with greater clearness in Figure 11, where the coefficients for the best rate (4.08 gallons per minute) and best total head (6 feet) are plotted for the three values of head-on-sprinkler which were tested. The curves on the diagram otherwise correspond to those of Figure 10. The dimensions of each disk are indicated in Figure 9.

There remains still to be considered the influence of the shape of the splashing disk itself. Two general patterns were studied — simple concave plates and disks of the "mushroom type" with a raised central cone (Nos. 13, 14, 17, 18, and 19). The latter show coefficients intermediate between the large and small concave disks, better than the former and not so good as the latter. In other words, the slope on one side of the central boss deflects the water very much as a simple cup of similar size and curvature would do. The discharge from these disks was, however, unequal along different radii, since it was almost impossible to make the stream of liquid strike the central boss. The coefficients are therefore higher than they should be, since the revolving trough caught portions of unequal discharges in

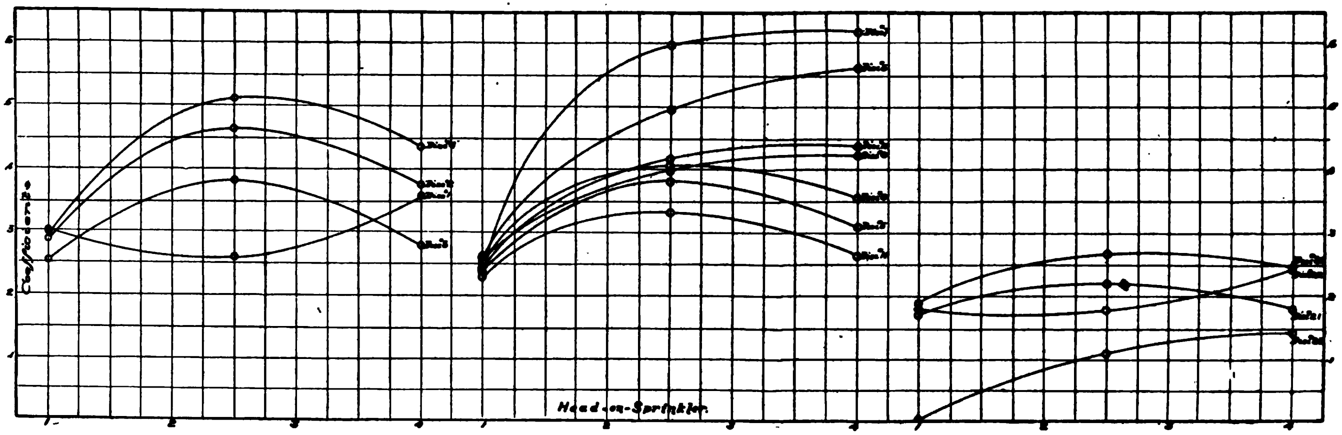

FIG. 11.—RELATION BETWEEN HEAD-ON-SPRINKLER AND COEFFICIENTS FOR DISKS OF DIFFERENT RADII OF CURVATURE
Rate = 4.08 gallons per minute. Total head = 6 feet.

different zones along the various radii. There seems to be no advantage in the use of disks of this pattern.

There remain the simple concave plates, varying in diameter and in radius of curvature. For diameter, 3 inches is clearly the most favorable size. A comparison of the three sets of curves in Figure 10 shows that the 3-inch disks (Nos. 5, 6, 7, 8, 9, and 10) are better than the 2-inch disks (Nos. 1, 2, 3, and 4), and far superior to the larger ones (Nos. 20, 21, 22, and 23). These figures, it must be remembered, relate to the 4.08-gallon discharge. At the lower rate (1.71 gallons per disk per minute) the 2-inch disks give higher coefficients than the 3-inch ones. This might be expected, since poor distribution with large disks arises from the fact that the liquid has not force enough to splash freely off, but pools and drips over the edge. The smallest disks are therefore best suited to low discharges, but the 3-inch disks and 4.08-gallon rate form the best combination.

The effect of varying radius of curvature is well shown in Figure 10 for the 4.08-gallon rate; and the constancy of the relation is indicated by the comparison made in Table VII.

TABLE VII — ORDER OF DISKS ACCORDING TO THEIR COEFFICIENTS (BEST DISK FIRST)

Rate, gallons per minute per disk.	Total head.	Diameter of Disks.		
		2 inches.	3 inches.	6 inches.
		Radii.	Radii.	Radii.
	Feet.	Inches.	Inches.	Inches.
1.71	2	4 6 12 20	2 4 4 12 6 20 20	20 12 6 4
	4	4 20 6 12	4 2 4 6 12 20 20	20 12 6 4
	6	4 6 20 12	2 4 12 6 20 20	6 12 20 4
4.08	2	4 6 12 20	2 4 4 12 6 20 20	6 20 12 4
	4	4 6 12 20	2 4 4 6 12 20 20	6 12 20 4
	6	4 6 20 12	2 4 6 12 20 20	12 6 20 4
8.47	2	4 6 12 20	2 4 4 6 12 20 20	6 4 12 20
	4	4 6 12 20	2 4 4 6 12 20 20	6 12 20 4
	6	4 6 12 20	4 2 6 20 20 12	6 12 20 4

Efficiency evidently increases with the increasing concavity of the disks. Disks Nos. 12 and 16, with very flat surfaces, gave such poor results that they have not been plotted. The improvement with increase in curvature is most marked with the disks of small diameter, and with the 6-inch disks the 12-inch radius of curvature is the maximum, the 4-inch radius giving very poor results. This is brought out more clearly in Figure 12, where the coefficients at the 4-foot head are plotted against the actual depth of the disks of various diameters. An increase in concavity increases the efficiency of the disk by breaking the liquid up more completely and sending it higher into the air. The accompanying increase of disk surface over which the liquid must travel tends, however, to decrease the efficiency of distribution, probably by the greater friction involved. With small disks this is not serious, but as the surface is increased this factor limits the possible increase in concavity. Disk No. 15, with a diameter of 5 inches and a radius of curvature of 3 inches, represents an extreme condition in which the liquid did not splash at all, but eddied about in the deep cup and merely dripped over the edge.

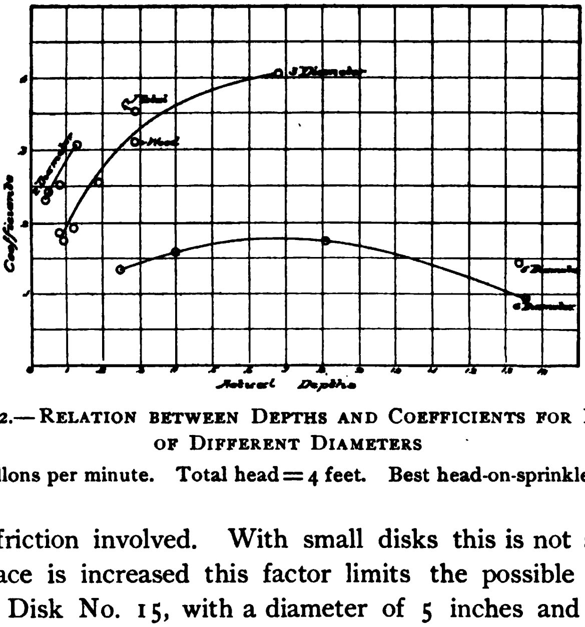

FIG. 12.—RELATION BETWEEN DEPTHS AND COEFFICIENTS FOR DISKS OF DIFFERENT DIAMETERS

Rate = 4.08 gallons per minute. Total head = 4 feet. Best head-on-sprinkler in each case

The only other point noted was the effect of the composition of the disks. Nos. 6, 7, and 15 were of metal, spun out of No. 18 sheet copper. All the others were of white pine, turned and carefully smoothed on the lathe, and varnished. No. 15, as noted above, was of very poor shape, but Nos. 6 and 7 gave the best results obtained in the whole series. No. 7 had a 2-inch radius of curvature, and should therefore have given better results than the others, which were of less concavity. No. 6, however, was of the same size and shape as No. 10 (of wood). Its coefficients were distinctly better under the most favorable heads. Thus at the 4.08-gallon rate, with a 6-foot total head and 2.5-foot head-on-sprinkler, No. 6 gave .49 against .42 for No. 10; with 4-foot head-on-sprinkler it gave .56 against .44.

In general, it appears from this portion of the investigation that the best results with gravity distribution can be obtained under the following conditions:

1. The discharge on each sprinkler should be in the neighborhood of 4 gallons per minute; this means, for a 2,000,000-gallon rate, 340 sprinklers per acre, with a distance between the sprinklers of about 11 feet.

2. The head between the distributing trough and the filter should be as great as possible; 2 feet is inadequate, 4 feet gives fair results, and 6 feet is better. Where exceptional conditions make still higher heads possible, the use of large, deep disks might prove of advantage.

3. The head on the sprinkler should be from 2 to 4 feet. The best subdivision of available total head can probably best be determined by experiments with the disks to be used in each individual case.

4. A simple concave disk of metal seems to promise the best efficiency.

5. The best diameter for the disks appears to be 3 inches. For low rates of discharge smaller disks are better, and for very high rates or very high heads larger ones may be more suitable.

6. Unless the disk be too large it is of advantage to increase its concavity as much as possible. Of 3-inch disks, that having a curvature corresponding to a radius of 2 inches proved most satisfactory. The radius of curvature might profitably be increased toward the limiting value of 1½ inches, which would make the disk a hemisphere. With larger disks larger radii of curvature are necessary.

Of all the disks studied, the best was No. 7, a metal cup, 3 inches in diameter, with a radius of curvature of 2 inches. With a total head of 4 feet and a head-on-sprinkler of 3 feet, it gave a coefficient of .40, the best obtained with that total head; and with a total head of 6 feet and a head-on-sprinkler of 4 feet, it gave the highest coefficient obtained for any gravity distributor, namely, .62.

V. DETERMINATION OF COEFFICIENTS FOR PRESSURE DISTRIBUTORS

The system of spraying sewage upward from fixed sprinkler nozzles under pressure was developed more or less independently at various plants in England. At Salford it was worked out after the failure of various other methods, in which the sewage overflowed from troughs, or was spread by a thin layer of sand covering the filter proper. In the earlier experiments with fixed nozzles, a disk-like cap was placed a little above each in order to break the sewage into a fine spray. Then the attempt was made to break up the stream by the impact of two converging flows; and finally a nozzle was designed which discharged the sewage from a series of openings arranged spirally about a centre. In Derbyshire, Barwise describes as common the use of fixed perforated pipes with metal disks placed over the outlet to form a spray (very much like the earliest Salford type). At Birmingham small sprinkler heads are used, in which the sewage passes up through a narrow, annular opening and is broken up and thrown outward by striking the bottom of a metal plug raised a little above it.

In America the first tests of trickling filter distribution were made at Columbus, Ohio. For the preliminary experiments of 1904–05 a nozzle was designed which recalls that used at Salford. The sewage was discharged through eight 1/8-inch brass pipes, at an angle of 45° with the vertical, into the space between an inner and an outer cone. The sewage left the nozzle in a tangential direction and spread in a fine sheet. Clogging proved too serious an objection to this nozzle, and another design was finally adopted. The latter was described by Mr. J. H. Gregory in the *Transactions of the American Society of Civil Engineers*, 57, 130, as follows: "The nozzle, which is of brass, consists of a single orifice, 9/16 inch in diameter, with rounded edges, above which, held by two thin arms, is an inverted 90° cone, the axis of the cone coinciding with the axis of the orifice. The jet, on leaving the orifice, impinges against the cone and is transformed into a thin sheet, spreading out radially and breaking into a shower of fine drops. At the points where the sheet strikes the arms the latter are ground down to sharp edges on each side, thereby permitting the sheet to heal up entirely after passing them."

At the Waterbury Experiment Station, Mr. W. G. Taylor has made a careful study of the general operation of sprinkler nozzles, and has devised still another type. He reports (*Engineering Record*, 55, 10) that a single opening with a cone above for distribution, supported at the sides as in the Columbus nozzle, proved unsatisfactory. The side supports broke the sheet badly, especially after the development upon them of a fungous growth. Furthermore, he points out that the "uniformity of distribution obtained by means of a single cone acting under a head of 5 feet was not satisfactory. Although the efficiency as marked by the area wetted was approximately 75 per cent., a certain outer zone invariably received sewage at a higher rate than the inner area. To lessen this defect, and to secure a greater percentage of wetted area and a more uniform distribution of the sewage, the idea occurred to the writer of placing a lower and secondary cone over the orifice, with the object of intercepting a portion of the sewage and distributing it over the inner fifth of the circular area within the range of the nozzle. The result was effected by boring a hole through the lower cone slightly less in diameter than the fixed orifice in the nozzle, the diameter of the opening in the cone being so proportioned to the diameter of the nozzle orifice that four-fifths of the rising stream passes through the lower cone to be diffused by the upper, and the remaining fifth is sheared off by the lower cone and sprinkled over the central area."

It is evidently desirable to have an accurate measure of the comparative performance of these sprinkler nozzles and of the still better designs which the future will probably develop. Mr. Gregory, in describing the Columbus experiments, was content to say that the nozzle used wetted 77 per cent. of the tributary area "in a perfectly satisfactory manner." Mr. Taylor noted, but did not measure, serious inequalities of distribution within the wetted area. We were therefore particularly glad to apply our method of measuring efficiency to the most promising of these pressure nozzles, and the results have proved significant. Through the kindness of Mr. G. C. Whipple, of New York, we have been furnished with two of the nozzles used at Salford and with the Birmingham type. To Mr. Gregory we owe the Columbus sprinkler, and Mr. Taylor has courteously given us drawings of the Waterbury design. The tests of these five types may best be discussed one by one, and reviewed in comparison with each other later.

The Salford nozzle (old style) is a brass nozzle in the form of a cup, with two openings in the sides at such an angle that the streams discharged upward from them meet in the cup itself. Its construction is shown in Figure 13.

This nozzle differed from all others tested in the fact that it did not discharge equally along different radii. It threw the liquid up in a fan-shaped spray, in quite a narrow plane, as indicated in Figure 14. Our ordinary method of rating efficiency, which assumes symmetrical distribution about the centre, was of course inapplicable under such circumstances. Instead, therefore, of revolving the measuring tank about this sprinkler, we measured the discharge at six different stationary positions, immediately adjacent to each other and together covering one-half of the entire circle surrounding the distributor. The average discharge in each zone was used in the manner described above for calculating the crude coefficient. Then the mean discharge along each sector was determined, and from this the total excessive discharge along those sectors which received more than their share. The ratio of the excessive radial discharge to the total discharge was then determined and taken from unity, giving the "radial coefficient." The

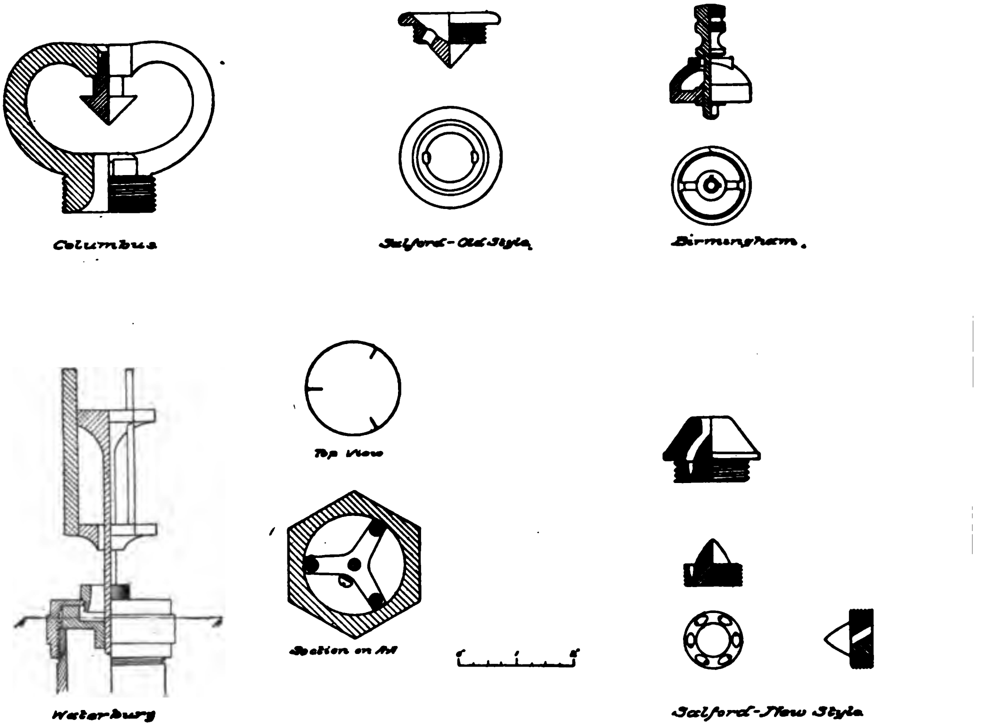

FIG. 13.—TYPES OF PRESSURE NOZZLES TESTED

product of the crude coefficient and the radial coefficient measures the evenness of distribution within the wetted area.

When this sprinkler nozzle was tested under a head of 6.4 feet, another anomaly appeared in connection with the determination of the corrected coefficient. The area wetted was larger than the square which would be tributary to each sprinkler at a 2,000,000-gallon rate.

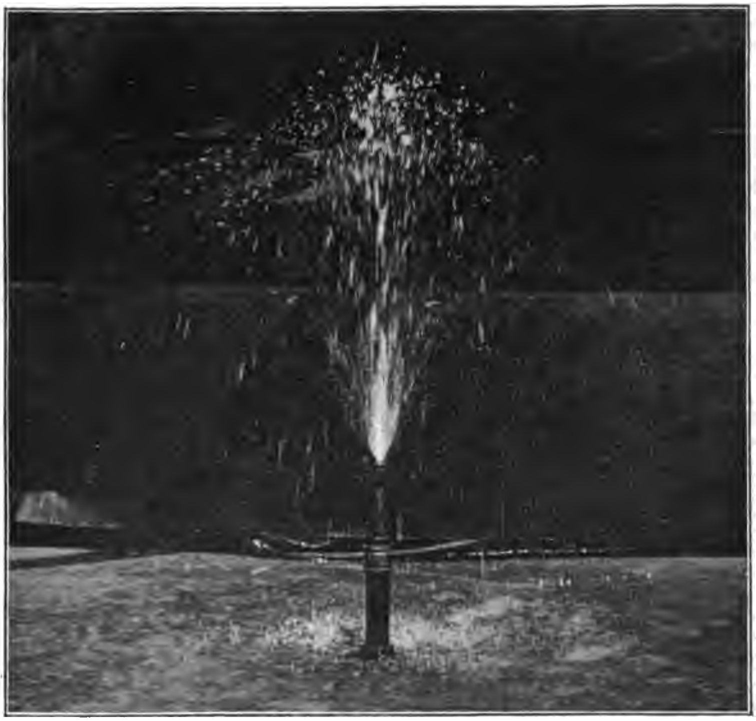

FIG. 14.—VIEW OF SALFORD OLD STYLE NOZZLE IN OPERATION

This means, of course, that at this head and rate the circles would overlap. The following method of calculation was applied in this case and in others of the same kind.

The effect of overlapping circles of spray is exactly the same as it would be if each portion of the spray passing the boundary of the tributary square were perfectly reflected at the side of that square and returned to a corresponding point within. It is necessary, therefore, to calculate how much of the discharge into the outer rings would have been thus reflected, or, in other words, what proportion of each of the overlapping rings falls without the square. Given the dimensions of the square and of each ring, this is a simple problem in trigonometry, the solution of which gives,

$$\frac{P}{100} = \frac{\cos^{-1}\frac{S}{D}}{45^\circ},$$

in which S is the side of the square and P the percentage of the total discharge into any overlapping ring of diameter D, which would be reflected to an inner ring. Having subtracted from all the rings the amounts which would be reflected, it is next necessary to distribute these amounts among the inner rings where they properly belong. The mathematical solution of this problem gave a result too cumbrous to use. A graphical representation of the conditions drawn on plotting paper, however, indicated readily and with sufficient accuracy the amounts of the reflected discharge which would fall into each of the inner rings affected, and the reflected water was then redistributed. These new values were then recalculated like the others and a coefficient obtained, which referred, not to a circle, but to the area common to the maximum circle and the intersecting square. The relation between this area and that of the square furnished the required factor for the calculation of the corrected coefficient from the crude.

The results of the tests on the Salford old style nozzle are shown in Table VIII. The total head, in this and succeeding tables, is the difference in level between the surface of the liquid in the supply box and the top of the measuring tank. The head-on-sprinkler is the difference in level between the surface of the liquid and the nozzle itself. In general, the nozzle was placed as close as possible to the top of the tank.

One very interesting point about this nozzle is brought out in the column of rates. The discharge with a head-on-sprinkler of 6.2 feet was less than with a head-on-sprinkler of 3.9 feet. This is apparently due to the fact that the two converging streams discharged by this nozzle meet in its cup with such force as to produce a back pressure, which at high heads cuts down the flow appreciably.

TABLE VIII — COEFFICIENTS FOR SALFORD OLD STYLE PRESSURE DISTRIBUTOR

Total head.	Head-on-sprinkler.	Rate, gallons per minute.	Coefficients.			
			Crude.	Radial.	Product.	Corrected.
Feet.	Feet.					
2.0	1.9	2.6	.65	.56	.36	.03
4.1	3.9	3.8	.73	.53	.38	.16
6.4	6.2	2.9	.76	.58	.44	.41

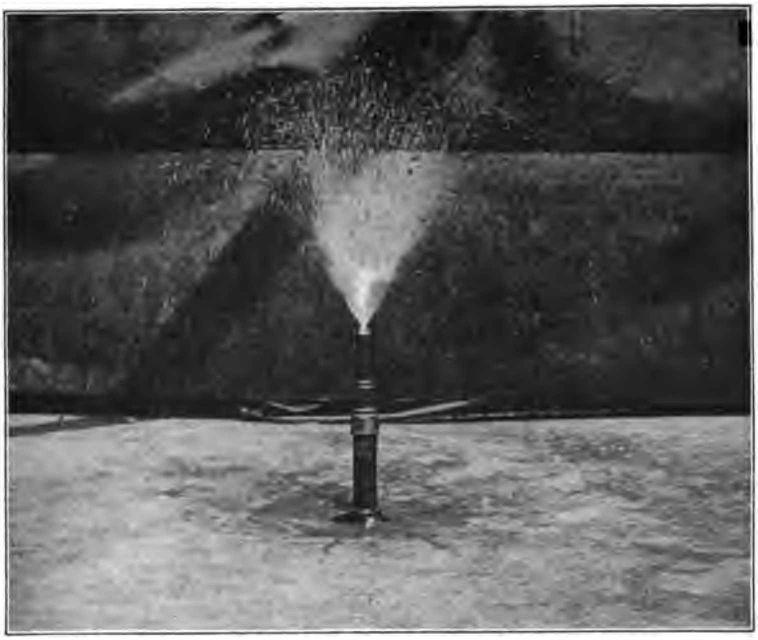

FIG. 15.— VIEW OF SALFORD NEW STYLE NOZZLE IN OPERATION

It is evident from the column of crude coefficients that this nozzle secures a fairly even distribution within the wetted sector, improving with the head to a very high value. There is an area of excessive discharge near the outer limit of the wetted sector, but the excess is not great. The inequality of distribution along different radii is much more serious. Nearly half the liquid is discharged in excess on one of the six 30° sectors, and this weakness causes the products and corrected coefficients to be poor.

The Salford new style sprinkler tested is a double cone, the outer cone being open at the top. The liquid enters the space between the cones through six holes of $\frac{5}{32}$ inch diameter, inclined at an angle to the major axis, so that a spiral motion is produced within the inner cone. This sprinkler is shown in Figure 13, and its general method of operation is indicated by the photograph reproduced in Figure 15.

With this, as with the other pressure nozzles, distribution along different radii was approximately equal, and tests were made in the manner described in Section II by revolving the measuring tank about the distributor. Duplicate determinations were made at each of the three heads tested, and the results are shown in Table IX. As in the case of the old Salford nozzle, at high heads a wetted area was obtained extending beyond the sides of the calculated tributary square, and the factor was therefore corrected by the method described above (page 359). The crude coefficients refer in all cases to the actual wetted circles.

TABLE IX—COEFFICIENTS FOR SALFORD NEW STYLE PRESSURE DISTRIBUTOR

Total head.	Head-on-sprinkler.	Rate, gallons per minute.	Crude coefficient.	Corrected coefficient.
Feet.	Feet.			
2.2	2.2	0.7	.37	.22
2.2	2.2	1.2	.44	.16
4.3	3.8	1.6	.68	.39
4.3	3.8	1.7	.70	.38
6.4	6.1	2.0	.78	.67
6.4	6.1	2.1	.77	.65

The results of these tests indicate that the new Salford sprinkler gives very good results at the higher heads. The tests themselves show an excessive discharge on the outer part of the wetted area, but the excess is small, giving crude coefficients of .6 and .7 at 3.8-foot and 6.1-foot heads. At the 3.8-foot head, however, the wetted area (a circle of 3 feet radius) is small in proportion to the rate. The corrected coefficient is therefore poor, while that obtained at the 6.1-foot head, with only slightly higher rate and a circle of 4 feet, remains in the neighborhood of .7.

The Birmingham nozzle, shown also in Figure 13, discharges through a $\frac{3}{8}$-inch opening. Through this opening passes a $\frac{7}{32}$-inch shank, the enlarged upper portion of which breaks up the ascending ring of liquid and sends it out in a fine spray. The working of the nozzle is shown in Figure 16, and its results are indicated in Table X. In this case the discharge through the opening was so small and the distribution so good that a circle larger than the proper tributary square

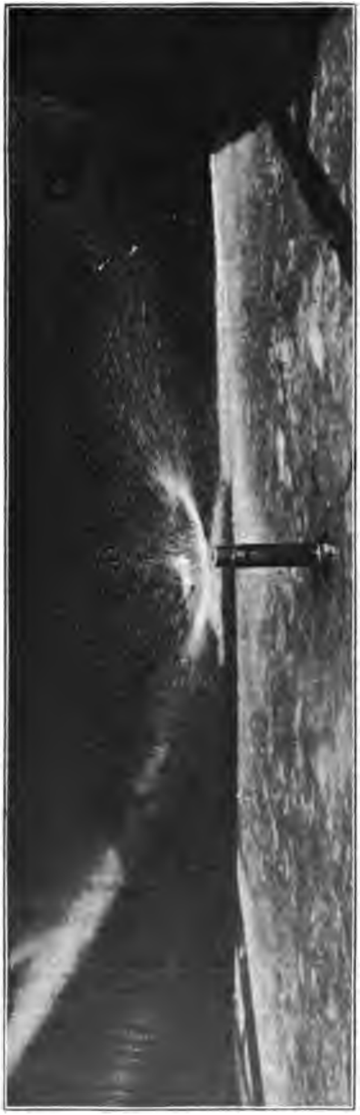

Fig. 16.—View of Birmingham Nozzle in Operation

was wetted in every test except those made at the 2.2-foot head. The results are therefore corrected, as in the cases above.

TABLE X — COEFFICIENTS FOR BIRMINGHAM PRESSURE DISTRIBUTOR

Total head.	Head-on-sprinkler.	Rate, gallons per minute.	Crude coefficient.	Corrected coefficient.
Feet.	Feet.			
2.2	2.0	0.9	.58	.44
2.2	2.0	1.0	.62	.38
4.3	4.1	1.4	.80	.80
4.3	4.1	1.5	.76	.72
6.4	5.8	1.8	.84	.69
6.4	5.8	2.0	.78	.66

These figures are of the greatest interest as showing what results may be obtained with a nozzle of small bore so arranged as to break up the stream quickly and completely. With a low rate of discharge, due, of course, to the obstructive construction of the orifice, a relatively large circle was covered (of 2.5 feet radius at the 2-foot head, 4 feet radius at the 4.1-foot head, and 5 feet radius at the 5.8-foot head); and within this circle the distribution was remarkably even. With this nozzle the 4-foot head proved better than the higher one, since at the 5.8-foot head a more definite ring of excessive discharge was formed. Under the best conditions this nozzle gave corrected coefficients of .7 to .8. It is clear that the Birmingham nozzle effects a far more perfect distribution than any other device we have examined.

The Columbus nozzle is designed on a plan exactly opposite to that of the Birmingham sprinkler, in so far as it aims to enlarge the orifice of discharge as much as possible. The Columbus design has been described above, and is also shown in Figure 13. Figure 17 illustrates its method of operation.

TABLE XI — COEFFICIENTS FOR COLUMBUS PRESSURE DISTRIBUTOR

Total head.	Head-on-sprinkler.	Rate, gallons per minute.	Crude coefficient.	Corrected coefficient.
Feet.	Feet.			
2.2	2.2	7.1	.54	.12
2.2	2.2	7.1	.60	.14
4.3	4.3	10.9	.63	.21
4.3	4.3	10.9	.65	.22
6.4	6.4	14.8	.61	.30
6.4	6.4	15.8	.62	.26

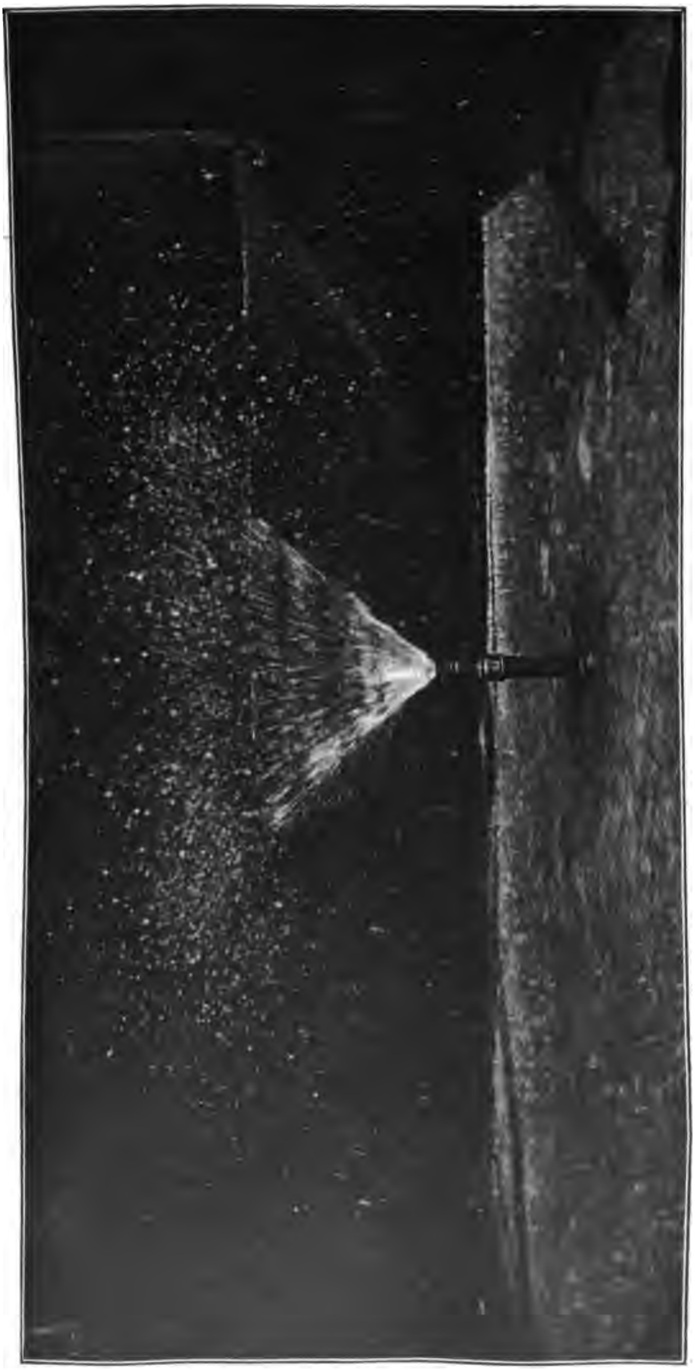

FIG. 17.—VIEW OF COLUMBUS NOZZLE IN OPERATION

The Columbus sprinkler shows at all heads a fair distribution within the wetted area, its departure from the ideal condition being due to the formation of a zone of excessive discharge a little inside the periphery of the wetted area. Crude coefficients, however, are always about .6, and one notable thing about this nozzle is the fact that it gives fairly even distribution even at the 2-foot head. The wetted area is, however, small in proportion to the discharge, and therefore the corrected coefficients are poor. Thus at the 4.3-foot head, for example, the rate of discharge of the nozzle was 10.9 gallons per minute. This discharge, if the total rate on the filter be 2,000,000 gallons per acre, calls for 121 sprinklers per acre, which means a distance of 19 feet between sprinklers. The wetted circle at this head was only 6 feet in radius, so that at an ostensible 2,000,000-gallon rate half the filter area would be working at 4,000,000 gallons and the rest would be idle. It is no doubt for this reason that the Columbus engineers plan to operate at a 4,000,000-gallon rate for half the time, and to rest the whole filter the remainder of the time, spacing their nozzles 16.2 feet apart. This would of course give a distribution more nearly approaching our crude coefficients, though the results at best would be below .6.

The Waterbury nozzle used in these experiments was of the type described on a preceding page (356), in the citation from Mr. Taylor's paper. Its construction is indicated in Figure 13. Briefly the nozzle consists of two cones, one above the other, the lower one perforated at its centre. An annular discharge is in part deflected outward by the lower cone, while its inner portion passes through the lower cone and is deflected by the upper one. As shown in Figure 18, the result is to produce two concentric cones of spray and thus improve the distribution obtained with a nozzle of the Columbus type. In the sprinkler tested by us, the position of each cone in relation to the point at which the liquid leaves the pipe below could be varied by means of set screws, and a number of different settings of these cones were examined. The distance between the lower opening and the lower surface of the lower cone at its outer rim is designated by the letter *a*; the distance between the upper surface of the lower cone and the lower surface of the upper cone is designated by the letter *b*. Reference to Figure 13 will make the significance of these dimensions clear.

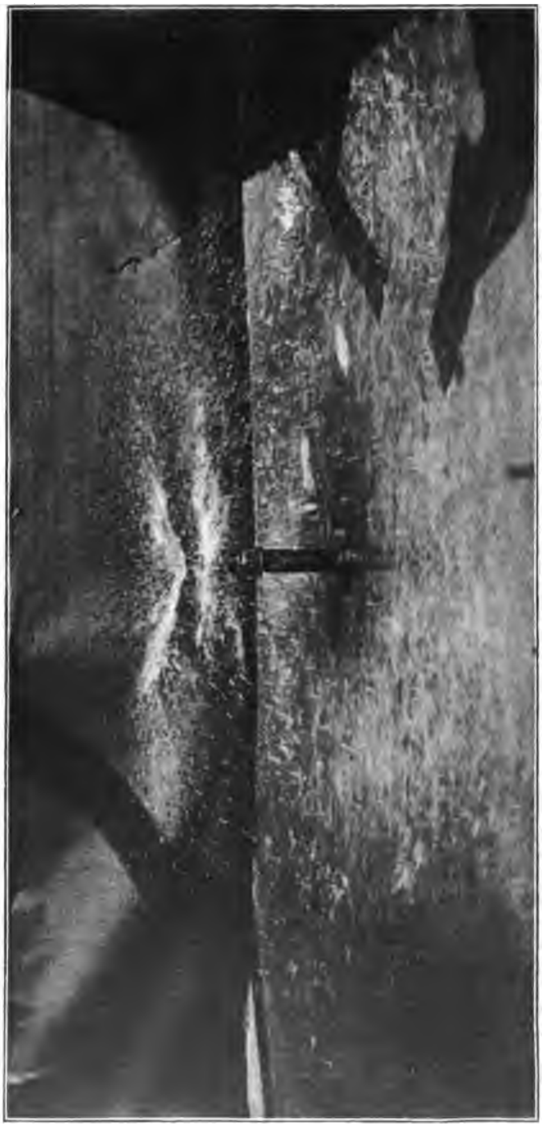

FIG. 18.—VIEW OF WATERBURY PERCONATE NOZZLE IN OPERATION

TABLE XII — COEFFICIENTS FOR WATERBURY PRESSURE DISTRIBUTORS

Total head.	Head-on-sprinkler.	Rate, gallons per minute.	HEIGHT OF CONES.		Crude coefficient.	Corrected coefficient.
			a.	*b.*		
Feet.	Feet.		Inches.	Inches.		
2.2	2.2	5.4	1⅜	1¾	.46	.05
2.2	2.2	5.6	1⅜	1¾	.45	.05
4.3	4.3	7.6	¾	1¾	.53	.11
4.3	4.3	7.8	¾	1¾	.62	.13
4.3	4.3	7.5	1⅜	1⅛	.62	.13
4.3	4.3	7.7	1⅜	1⅛	.62	.14
4.3	4.3	7.6	1⅜	1¾	.62	.14
4.3	4.3	7.7	1⅜	1¾	.68	.14
4.3	4.3	7.8	1⅜	1¾	.66	.13
6.4	6.3	6.8	¾	1¾	.68	.21
6.4	6.3	10.2	¾	1¾	.72	.21
6.4	6.2	8.6	1⅜	1⅛	.62	.15
6.4	6.4	9.6	1⅜	1⅝	.69	.22
6.4	6.4	10.4	1⅜	1⅝	.73	.22
6.4	6.2	9.3	1⅜	1¾	.77	.21

The results of the tests on the Waterbury sprinkler are indicated in Table XII. It will be noticed that the rates varied somewhat under the same conditions. This was no doubt due to the tendency of this sprinkler to clog. Our tests were all made with water, but the tanks through which it was pumped were not perfectly clean and at times contributed their quota of suspended solids (old sewage sludge). These inequalities did not, however, seriously affect the coefficients, which are fairly uniform. It appears from this table that the relative position of the cones does not greatly modify the efficiency of distribution, although there is a slight but distinct improvement in coefficients with an increase in the value of *b*. That is, an increase in the distance between the upper and lower cone improves distribution by throwing the outer cone of liquid further out and separating it more distinctly from the inner one.

In all cases, however, there is more or less fusion of the two cones of spray, and our tests showed a single zone of excessive discharge, not near the periphery of the wetted area, as with the Columbus nozzle, but about a third or a half of the way out from the centre of the wetted circle. The crude coefficients at 4.3-foot and 6.4-foot heads were a little higher than those obtained with the Columbus nozzle, averaging a little over .6 for the lower and about .7 for the higher head. The wetted area with this nozzle is, however, small for its high rate of discharge. The largest circles wetted were 11 feet in diameter, and this was at rates of about 10 gallons per minute, corresponding to distances of 18 feet between sprinklers. The corrected coefficients are therefore low, the best of them being barely over .2.

Mr. Taylor has worked out certain modifications of his sprinkler which have not been examined in the present investigation, but which well deserve a careful study. Two of these are described in the paper previously cited (*Engineering Record*, 55, 10) and more fully in a somewhat later one (*Engineering News*, 57, 238). One consists of a six-winged revolving cone supported directly over a single orifice, through which the sewage rises. The six blades, shaped like air blowers, are rotated by the force of the liquid striking them, and their curvature gives the drops of sewage, leaving them a widely varying angle of elevation. A still more interesting nozzle has a cone similar to that tested in our experiments, but with a vertical cut in each quadrant of the cone. The upper surface of the conical piece has almost the shape of a four-leaf clover, though not cut in so deeply, and the sewage is discharged from the outer rounded corners at a more oblique angle than from the incised areas between. It is hoped that by the proper construction of a cone of this type "the sewage film when leaving shall have such varying angles of elevation that it shall, upon diffusion, throw the proper proportion of drops the distance required to cover the side of the square, while midway between these points, on the diagonal lines, the drops will, by reason of their greater angle of elevation, be thrown with the tendency to fill the corners of the square."

VI. THE EFFICIENCY OF PRESSURE DISTRIBUTORS WITH VARYING HEADS

With a sprinkler which, like many pressure nozzles, discharges in a conical sheet and forms an excessive discharge on a peripheral zone, it would seem to be of advantage to vary the head-on-sprinkler in such a way as to cause the zone of excessive discharge to travel in and out over the wetted area. This may easily be done by the interposition of siphon tanks in the supply system, and such is the practice at many of the English plants, notably in Derbyshire (Barwise, "Purification of Sewage," London, 1904). The Board of Advisory Engineers to the

Baltimore Sewerage Commission have suggested a similar arrangement for their projected trickling filters (Report of the Board of Advisory Engineers and of the Chief Engineer, Baltimore, 1906). At Columbus, Mr. Gregory, in the paper previously cited (*Transactions of the American Society of Civil Engineers*, 57, 130), states that the sprinklers will be operated at a head varying during twenty-four hour periods from 2.75 feet (a rate of 10 gallons per minute per sprinkler) to 5 feet (a rate of 13.5 gallons). We have had a Columbus nozzle with varying head in operation on the trickling filter at the Technology Experiment Station since the summer of 1906 with apparently good results, and the Massachusetts State Board of Health has used a siphon tank in connection with their gravity distributor at Andover (*Engineering News*, 57, 397).

It is clear that such varying heads must tend to improve the efficiency of devices like the Columbus sprinkler. Just how much improvement is effected under such conditions can, however, only be determined by exact measurement. We have therefore made a study of the operation of the Columbus sprinkler at varying heads, and the results have proved of considerable interest.

We used in our series of tests a maximum head of 4 feet (in two cases 3.7 feet) and a varying minimum head from 3.5 down to 2 feet. The discharge was made from a square siphon tank equipped with float valves so as to start and cut off sharply. The time of filling this tank varied from thirty-seven seconds to three minutes and forty-eight seconds, and the time of its discharge from twenty-seven seconds to two minutes and forty-seven seconds.

The rate in all cases was in the neighborhood of 4 gallons per minute. It was somewhat higher than this in some tests, but the wetted area was in all cases a circle of 10 to 10.5 feet diameter, and the rates corresponded with sufficient closeness to a spacing of nozzles 11 feet apart, on which basis all have been calculated. The coefficients are therefore comparable with those given for a constant head of 4.3 feet in Table XI. These results are reproduced for comparison in Table XIII.

TABLE XIII—COEFFICIENTS FOR COLUMBUS DISTRIBUTOR WITH VARYING HEAD

Maximum head.	Minimum head.	Crude coefficient.	Corrected coefficient.
Feet.	Feet.		
4.3	4.3	.63	.21
4.3	4.3	.65	.22
4.0	2.0	.45	.30
4.0	2.5	.50	.33
4.0	3.0	.56	.37
4.0	3.0	.56	.37
4.0	3.2	.53	.41
4.0	3.5	.53	.41
4.0	3.5	.55	.43
3.7	3.5	.49	.32
3.7	3.5	.48	.38

It was expected, *a priori*, that a varying head would give more even distribution within the wetted area; and to the eye the waxing and waning cone of spray looks well-nigh perfect. Nothing could better illustrate the importance of the rigorous method of measurement adopted in these experiments. The crude coefficients in Table XIII are distinctly poorer with varying than with constant heads. An inspection of the original measurements shows the presence of a well-marked zone of excessive discharge just as in the case of the constant head, only not so far out from the centre of the wetted area. A large excess of the liquid was found in the compartments 2 and 2½ feet out from the nozzle. The smaller radial distance of this maximum zone accounts for its greater effect in reducing the coefficient.

On the other hand, the corrected coefficients were much improved by the intermittent operation. The wetted area was as great as in the tests at a constant head of 4 feet and the discharge was much less. The average rate was as low as 4 gallons per minute per sprinkler. Therefore, for a total rate of 2,000,000 gallons per acre the nozzles could be placed much closer than at a 4-foot constant head (11 feet instead of 19), and the wetted area made to approximate more nearly to the tributary squares. Thus the corrected coefficients for a head varying from 4 feet down range from .3 to .4 against a little over .2 for the constant head of 4.3 feet.

With regard to the best construction for a siphon tank, Table XIII brings out several suggestive points. Taking the maximum heads of

4 feet, it is apparent that the evenness of distribution within the wetted area is better when the tank is not too long in emptying, since the crude coefficients are better with a minimum head of 3 feet and over than with lesser ones. As the minimum head is increased, the corrected coefficient rises still more rapidly, increasing progressively from .30 to .43. This is because the wetted area at 3.2 feet and 3.5 feet minimum heads was somewhat larger than with a lower minimum, while the evenness of distribution was as good as at a 3-foot minimum head. Evidently the best siphon tank is one with a short range of level which will discharge a small volume of sewage at frequent intervals. In our tests the best arrangement was such that the tank filled and emptied in about one minute, the change of head being from 4 feet to 3.5 feet. The decreased maximum head in the last two lines of the table shows an injurious effect on both crude and corrected rates.

The siphon tank used in these experiments was a simple box of square section, but Mr. Taylor (*Engineering Record*, 55, 10) has suggested a design which promises better results. He points out that on theoretical grounds a nozzle acting under a straight-line variation of head from maximum to minimum, such as that produced by the discharge of an ordinary tank under siphon action, could not lead to perfect results. To produce ideal distribution the head upon the nozzle should vary less rapidly near the maximum point than near the minimum. He found that the rate of the desirable variation was represented by a parabolic line.

VII. COMPARATIVE EFFICIENCY OF DISTRIBUTORS OF VARIOUS TYPES

The optimum conditions for distributors of various types have been somewhat fully discussed, but a brief comparative summary of their performance may be of interest in closing. We have therefore summarized the results under optimum conditions (4-foot total head for Birmingham, 6-foot for the others) in Table XIV.

TABLE XIV—COMPARATIVE EFFICIENCY OF DISTRIBUTORS OF VARIOUS TYPES

Type.	Rate, gallons per minute.	Crude coefficient.	Corrected coefficient.	Sprinklers per acre to discharge 2,000,000 gallons per day.
Best gravity distributor	4.08	.76	.62	341
Old Salford	2.9	.44	.41	483
New Salford	2.1	.78	.67	667
Birmingham	2.0	.80	.80	700
Columbus	14.8	.61	.30	94
Waterbury	10.4	.73	.22	134

The American pressure nozzles, judged on the basis of these measurements, show a low efficiency in distribution. They cover the wetted area fairly well under the best conditions, but their discharge is so excessive that at a total rate of 2,000,000 gallons per acre they leave large waste areas between, and their corrected coefficients are correspondingly low. It must, of course, be remembered that at a rate of 4,000,000 gallons per acre, for which the Columbus nozzle at least is designed, the corrected coefficients would be much higher. Furthermore, the performance of these nozzles may be further improved by the use of varying head, which increases a coefficient of .2 (for 4-foot total head) to .4 (for a head varying between 3.5 feet and 4 feet). This question of varying heads well deserves further study, especially in the light of Mr. Taylor's plan for parabolic variations. The Columbus nozzle offers great advantages in the simplicity of its construction and in its large orifice if its excessive discharge can be overcome by the use of siphon tanks.

The best of the gravity distributors (a 3-inch metal disk with a 2-inch radius of curvature) has given much better results than we had hoped for it. Without the complication of siphon tanks and varying head, it shows a corrected coefficient of .62, double that obtained for the Waterbury and Columbus nozzles under the same conditions. With a 4-foot total head its coefficient is .40, while the Columbus nozzle gives .22 with a similar constant head, and the best Columbus nozzle value with a varying head is only .43. The smallest opening for clogging in the gravity system is the discharge pipe, ¾ inch in diameter, while the diameter of the Columbus nozzle is $\frac{9}{16}$ inch. Furthermore, the accessibility of the whole distribution system makes its maintenance in good condition easy.

The new Salford and Birmingham pressure nozzles produce the most efficient distribution of any of the devices tested. At a 6-foot total head the Birmingham coefficient is .69, and at a 4-foot total head it is still better (.80), the distribution being almost perfect. These good results are, however, gained by the use of nozzles which are extremely liable to clogging. The openings of the new Salford nozzle are only $\frac{5}{32}$ inch in diameter, and the cone into which they discharge is admirably adapted to collect solid materials. The opening in the Birmingham nozzle is a ring only $\frac{5}{64}$ inch in diameter. Its removable plug indicates that even with the carefully screened and septicized sewage of Birmingham it requires frequent attention; and it is difficult to see how any such device could be economically used under American conditions, even with septic sewage. On the other hand, crude sewage can be sprayed by the Columbus nozzle or the gravity distributor without difficulty from clogging.

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Contribution from the Sanitary Research Laboratory and
Sewage Experiment Station of the Massachusetts
Institute of Technology

THE STERILIZATION OF SEWAGE-FILTER EFFLUENTS

By EARLE B. PHELPS and WILLIAM T. CARPENTER

The Desirability of Sterilization

The primary purpose of sewage purification is the protection of the purity of the streams. The Lawrence experiments first showed that, by means of sand filters, sewage could be purified to an extent which is for all practical purposes complete. This means that practically all the organic matter and the living bacteria of the sewage are either removed or converted into harmless mineral constituents. Such a degree of purification is possible, but it involves the use of such large areas of suitable sandy land that, except under the most favorable circumstances, its cost to a city of moderate size is practically prohibitive. Consequently the tendency for the past decade has been toward more rapid processes of purification; that is, processes requiring smaller areas of land for the treatment of given volumes of sewage. The results of experiments along this line, carried on in many parts of the world, are now well known. We have seen the successive development of the septic tank, the contact filter, and the trickling filter, each of which has resulted in an increase in the possible rate of treatment, until the original maximum rate of fifty or sixty thousand gallons per acre per day has been multiplied fifty fold or more. This great saving in filter area has not been obtained without some sacrifice. With the progressive increase in the rate of treatment there has come a more or less progressive deterioration in the quality of the effluent, so that the effluent of a modern trickling filter, so far as appearance goes, and indeed often in its chemical analysis, seems to be little better than the crude sewage. But there is one essential difference. In passing through the filter the sewage has been robbed of its putrescibility, that is, of its tendency to undergo offensive putrefactive fermentation. The organic matter of the sewage is unstable, or liable to putrefy, while that of the effluent is stable. And we have learned that in bringing about this one change in the character of the organic matter we have accomplished the most essential thing in sewage purification, as will be seen if we analyze a little more in detail the effects of stream pollution. These may be conveniently considered under three heads—æsthetic, economic, and sanitary.

The discharge of a relatively small amount of sewage into a stream brings about no serious physical conditions. As the amount of sewage increases year by year, however, there is ever increasing evidence of its presence. The waters become discolored, a greasy scum appears on the surface, and, worst of all, the black mud deposited upon the bottom ferments, giving rise to malodorous gases. Finally, if the sewage be added in still greater amounts a point is reached, quite suddenly, at which the balance swings, and the stream becomes permanently black and offensive and will no longer support fish or other life. It has, in fact, changed from a polluted river to a sewer. There may be no thought of using this stream for domestic purposes, and there may even be no serious monetary damage involved, but there can be no more potent argument for sewage purification than such a ceaseless offence to the eyes and nostrils of the community. It is now a recognized fact that to prevent such conditions as these it is not necessary to free an effluent from organic matter, much less from bacteria, but that it is alone sufficient to render it thoroughly stable.

There are also certain definite economic damages caused by stream pollution. Communities are ofttimes under the necessity of using for sources of domestic water supply the waters of polluted streams. Still more frequently such sources are much more available than a more distant unpolluted source, and it is found to be more economical to filter the polluted water than to bring in the more distant one. No contention can be made that streams should be so far protected that they may be used directly for public water supplies. In the very nature of the case, streams flowing through populated regions are polluted, and the complete purification of the cities' sewage would still leave countless small but significant sources of possible infection. Therefore, the contention that it is not the duty of the sewage works to turn out a potable water seems reasonable. All ends are served most fairly and most economically by placing upon the water filter the burden of producing an unpolluted water for domestic use. It must be obvious, however, that the condition of gross physical nuisance described above as an open sewer rather than a polluted river must render it impractical, if not impossible, to treat such a water upon a water filter in the ordinary manner. It is equally true, though less obvious, that any considerable pollution of a stream by putrescible organic matter increases the work of the water filter and, in consequence, reduces its factor of safety and efficiency in the removal of disease germs. This contention will become more obvious if we bear in mind the fact that putrescible organic matter is matter requiring oxygen to render it stable, and that the work of the water filter is simply the oxidation of all such matter, together with the organic matter of the living bacteria and other organized animal and vegetable forms. To offset, therefore, this increased pollution lower rates of filtration and larger areas are necessary, with consequent increased expense for construction and maintenance.

Among other financial losses it is sufficient to mention the harm done to profitable fishing, decreased value of the water for industrial purposes, depreciated land values, and other monetary losses due to causes described as æsthetic.

In any equitable solution of the problem we must seek the middle ground, and carefully balance the cost of purifying sewage to any given degree of purity against the saving in filter area and maintenance thereby made possible on the water plant, and against other economic advantages. Experience has shown that non-putrescible effluents can be obtained from sewage filters run at feasible rates, and that the discharge of such effluents, if reasonably free from suspended matter, will throw little or no additional burden upon the water filter; that raw sewage or putrescible effluents reduce the power of the stream to maintain its own normal character and render its subsequent purification more difficult; and that the benefits of any more complete purification of the sewage beyond the point of perfect stability will be entirely incommensurate with their cost. Therefore the economic as well as the æsthetic arguments for the purification of sewage are met by such a treatment as will produce non-putrefactive effluents.

Sanitary considerations are themselves economic in their ultimate analyses, but there are certain aspects of the present subject which are, primarily, sanitary. The relation of sewage disposal to water filtration has already been commented upon. In the discussion of the economic phase of this relation it was assumed that satisfactory purification of the water would be obtained at whatever cost. Where polluted waters are being taken directly into the city mains the question is naturally a serious sanitary one, but again it must be asserted that the responsibility rests upon that community using the water. The most efficient purification of the sewage before discharge will never make a water safe which drains a populated country. The sewage can and should be rendered non-putrescible, but it will still be germ-laden. The removal of these residual germs, together with those others which by scores of devious and hidden paths have found their way into the stream, must depend upon the water works authorities. The case is aptly put by that dictum ascribed to Mr. Hering: "Nothing to be discharged into a stream without purification; nothing to be taken from a stream without purification." The greatest good to the greatest number finds its solution in this system of dual responsibility.

Sewage disposal has, therefore, curiously enough come to be less and less a matter of sanitation. As the practice has gradually developed from the slow sand filter toward rapid, coarse-grained filters, it has become more obvious that the removal of bacteria is not one of the functions of a sewage filter. There is, as a rule, a removal of a portion of these organisms. Whether or not there is a satisfactory elimination of the dangerous germs of disease is a point upon which there is great diversity of opinion and very little exact knowledge.

It was seriously feared at one time that pathogenic germs might multiply in the septic tank. Woodhead made physiological tests of crude sewage and septic tank effluent, and concluded that there were no organisms in the latter capable of setting up morbid changes in animals after inoculation. Rideal states that "satisfactory evidence in most of the systems is now available, from which I think we are justified in concluding that, even if towns on a river like the Thames adopted bacterial schemes, the pathogenicity of the London water supply would not be adversely affected." (Great Britain, 1902, A, Quest. 4148.)

On the other hand, Houston, acting for the Royal Sewage Commission, made a careful bacteriological study of the whole problem. He concludes that:

The effluents from septic tanks, intermittent contact beds, continuous filtration beds, etc., contain an enormous number of bacteria. In some cases the percentage reduction of microbes in effluent as compared with raw sewage is striking; but as an effluent must be judged by the actual state it is in, and as the number of microörganisms still remaining is almost always very large, percentage purification would seem to be of minor importance. In not a few cases the bacteria are practically as numerous in the effluent as in the raw sewage.

The different kinds of bacteria and their relative abundance appear to be very much the same in the effluents as in the crude sewage. Thus, as regards undesirable bacteria, the effluents frequently contain nearly as many *B. coli*, proteus-like germs, spores of *B. enteritidis sporogenes* and streptococci, as crude sewage. In no case, seemingly, has the reduction of these objectionable bacteria been so marked as to be very material from the point of view of the epidemiologist. No definite proof has been furnished that the effluents from bacteria beds are conspicuously more safe in this sense in their possible relation to disease than is crude sewage. Indeed, all the available evidence tends to show that they must be regarded as nearly if not quite as dangerous to health as raw sewage. In this connection we would again take note of the streptococcus test. If it be true that streptococci are more delicate germs than the typhoid bacillus, their presence in any number in the effluent would seem to indicate the possibility or probability of the enteric fever bacillus also surviving under similar conditions, and, in general, would lead us to infer that the biological processes at work were not strongly inimical, if hostile at all, to the vitality of germs of pathogenic sort. (Great Britain, 1902, B, p. 26.)

At the Sewage Experiment Station of the Massachusetts Institute of Technology, bacteriological examination of crude sewage and of the effluents of four septic tanks and fifteen filters were made during the summer season. The following average results were reported by Winslow (1904):

Sample.	Number of Examinations.	Bacteria per Cubic Centimeter.					
		Lactose gelatin at 20° C.			Lactose agar at 37° C.		Anaërobic agar.
		Liquefiers.	Acid formers.	Total.	Acid formers.	Total.	
Sewage	56	365,000	1,670,000	5,430,000	1,670,000	3,760,000	2,440,000
Septic effluent . . .	56	162,000	495,000	1,750,000	650,000	1,040,000	930,000
Contact filters . . .	140	60,000	270,000	1,060,000	290,000	570,000	440,000
Trickling filters . .	18	134,000	114,000	451,000	284,000	1,170,000	200,000
Sand filters	15	500	1,360	9,160	11,400	43,600	1,200

There were four septic tanks running at storage periods of from twelve to forty hours, nine contact filters differing in material, depth, and rate, three trickling filters, and three sand filters, each set running with crude and two kinds of septic sewage.

The septic tank and two trickling filters now in operation at this station gave the following bacterial results, the figures being averages of the results of almost daily examination during the past summer.

	Bacteria per c.c. Lactose agar at 37°.	B. coli. Positive* tests in $\frac{1}{1000000}$ c.c.
Sewage	1,300,000	65 per cent.
Trickling filter receiving sewage	750,000	35 per cent.
Septic effluent	1,650,000	66 per cent.
Trickling filter receiving septic effluent	750,000	35 per cent.

*The bile broth presumptive test, recommended by Jackson (1906), was employed.

About one-half the total organisms were removed by filtration, and the reduction of *B. coli* was practically in the same proportion. Such results as these would seem to justify fully the statement quoted from Houston.

We are still ignorant of the fate of the dangerous pathogenic organisms, so far as any direct experimental knowledge goes. In the absence of such knowledge it is only reasonable and safe to assume that such organisms do, in part, at least, pass through the filter.

We must therefore conclude that sewage purification by modern processes can be made to produce effluents which are reasonably clear and stable, but which are still germ-laden and, potentially, infectious; that the discharge of such effluents into streams or tidal waters will not bring about physical nuisance or economic damage; and that, so far as the use of water for domestic and industrial purposes is concerned, such discharge is just and reasonable.

At the present time, however, we are confronted with a consideration which promises to add a new chapter to the history of sewage purification. This is the question of the relation of sewage pollution and sewage disposal to the shellfish industry. Many of the oyster and clam beds of our eastern seaboard are seriously polluted with sewage. Both at home and abroad this subject has reached the magnitude of a "scare," and a serious falling off in the demand for these shellfish is reported. In this country, at least, most of this pollution is due to the discharge of untreated sewage into the rivers and tidal waters. In Great Britain it is more commonly the result of the discharge of partially purified effluents which are quite stable, as we have described them, but germ-laden. In many places the matter is being studied or agitated. In Great Britain the Royal Sewage Commission has issued a voluminous and valuable report upon the pollution of shellfish (Great Britain, 1904). In our own country the Massachusetts State Board of Health has had the question under investigation for several years, and has just issued a valuable report bearing upon the subject (Massachusetts, 1906); New York State has a Harbor Pollution Commission which for two years has studied conditions in New York Harbor; New Jersey is alive to the danger which threatens one of her great industries; and Baltimore is taking active steps to improve the sanitary conditions of upper Chesapeake Bay. The difficulties of the situation are obvious. A polluted river water may be rendered potable through filtration, but purification of shellfish seems to be out of the question. It follows, then, that the bacteria of sewage must not be discharged upon shellfish beds. The kind of purification which we have discussed will no longer serve, for sanitary considerations are here paramount. A higher standard of purity — a bacterial purity as well as an organic stability — must be demanded in such cases, or else the taking of shellfish must be prohibited. When the community is large and the shellfish industry small this latter alternative will probably prove the more satisfactory. Where small communities are polluting large and important beds, thorough purification by sand filters is not an unreasonable requirement. Unfortunately, the greatest shellfish areas of the country are so situated that they are subject directly or indirectly to possible pollution from concentrated populations.

In a carefully prepared paper on this subject by Mr. G. W. Fuller (1905) it is stated that the annual crop of oysters gathered along the Atlantic and Gulf coasts in 1902 amounted to over twenty-five million bushels, valued at over thirteen million dollars, and that the crop of clams was over two million bushels, valued at two million dollars. Over one-half of this total product came from the three states of New Jersey, Maryland, and Virginia, and were, in the main, grown in the waters of the Delaware and Chesapeake Bays, into which the sewage of Philadelphia, Wilmington, Baltimore, Washington, and other cities is discharged. At present, owing to the tremendous dilution of the polluting matters in these bays, the danger of pollution is in most cases remote, but it is significant, ever present, and ever increasing. The great value of the industry and the even slight danger to which it is now exposed has called for action on the part of those most interested. In Baltimore a system of sewerage and sewage disposal is being planned by which it is intended to remedy the present serious pollution of the Patapsco. In calling upon a commission of experts for advice, the Sewerage Commission of Baltimore distinctly specified "that the effluent proposed to be discharged into Chesapeake Bay or its tributaries in the system to be recommended by the engineers shall be of the highest degree of purity." By a coöperative arrangement between the states of New Jersey and Pennsylvania the condition of the Delaware River is being investigated, and measures will be considered by which the present serious pollution of that river shall be remedied.

It seems that in such cases the value of the industry, together with the difficulty of protecting the beds in any other way, will ultimately force upon the authorities one of two alternatives: complete purification by filtration through sand, or rapid and partial purification by means of contact beds or trickling filters, followed by some kind of sterilization.

The former is practically what has been recommended for the city of Baltimore, although it is there proposed to make use of sand filtration as a secondary process, following treatment on trickling filters. In this way the efficiency of the sand filtration is obtained at a considerable saving in filtration area. The Board of Advisory Engineers has estimated the cost of works for the complete treatment of seventy-five million gallons of sewage per day at $3,283,250, of which sum $1,040,750, or over 31 per cent., is for the supplementary treatment on sand filters. The annual cost of operation is estimated at $115,500, of which $55,000, or 48 per cent., is for the supplementary treatment (Baltimore, 1906). This gives some idea of the price which it is necessary to pay for effluents of bacterial purity over and above the cost of rendering the sewage stable and non-putrefactive. In regard to sterilization by chemicals, the engineers report that "to remove all bacteria remaining in the settled effluent from the sprinkling filters by disinfectants, such as hypochlorite of lime or of sodium or sulphate of copper, would be prohibitively expensive"; and this opinion is based upon the best knowledge of the subject now available. The fact is, this alternative — chemical sterilization — has not yet been reduced to its lowest terms. We have yet to learn how effective it may be made and at what cost, and what after effects, objectionable or otherwise, are likely to follow its employment.

A somewhat careful study of the question has forced the authors to the belief that there is much of value in this process, and that under certain circumstances and conditions which are common in this country, particularly where the oyster question is involved, it may be found to be the best possible solution of the whole problem.

In connection with an experimental study of certain kinds of chemical sterilization, the results of which are here presented, the literature of the subject has been thoroughly reviewed, and it is thought that a summary of our available information might prove of value. In connection with the experimental data which have been compiled, the all-important question of cost has been gone into in some detail, particularly in the case of those processes which in other respects appear to be feasible.

THE EFFICIENCY AND COST OF STERILIZATION

In Great Britain the somewhat indefinite allusions of the Royal Sewage Commission to the question of sterilization as a finishing process in sewage treatment have aroused a storm of discussion and resulted, at least, in clearing away many misconceptions. Sterilization processes of many kinds have been investigated, and the subject has been freely discussed. For the following classification of sterilizing agents and for many of the facts here cited the writers are indebted to Rideal (1905), whose careful paper on the subject discusses its possibilities in a thoroughly impartial manner. Other authorities are cited so far as possible. Except when otherwise stated, costs are based upon present Boston prices.

The various methods proposed for the sterilization of effluents will be taken up in the following order.

1. Heat.
2. Lime.
3. Acids.
4. Ozone.
5. Chlorine and its compounds.
    (*a*) Chlorine gas.
    (*b*) Oxychlorides.
    (*c*) Chloride of lime ; hypochlorites.

6. Copper and its compounds.
7. Miscellaneous.
   (*a*) Permanganate.
   (*b*) "Amines."

1. HEAT. — The use of heat has been suggested for the sterilization of sewage and effluents. In his testimony before the Royal Commission, Klein (Great Britain, 1902, A, Quest. 9674) referred to a patented apparatus which he considers quite feasible, and by the use of which enough ammonia was to be recovered nearly to pay for the fuel required. There is no record that this device has ever been used on a large scale, and its claims are almost too fanciful. A rough calculation of the anthracite coal necessary to raise sewage to the boiling point from the average temperature of 60° F. gives 40 tons per million gallons, which will cost say $4 per ton, or $160. The total amount of free ammonia in a million gallons of Boston sewage is about 100 pounds, which, if concentrated to the commercial strength of 28 per cent., would bring 40 cents per pound, or $40. These are the most favorable figures possible, and in reality the revenue would be a much smaller proportion of the cost. Sterilizing by heat, therefore, will be a very expensive process and may safely be disregarded.

2. LIME. — Caustic lime acts as a weak germicide. The considerable removal of bacteria noted when lime is used as a precipitant for sewage is undoubtedly due to the action of the precipitate itself in dragging down with it the bacteria which it has entangled. Such an action is common in all precipitation, and even the sedimentation of sewage is always accompanied by a great reduction in the numbers of bacteria. For use with effluents, therefore, lime alone would not seem to be a valuable germicide. Rideal states that sixty to seventy grains (850–1,000 parts) are inefficient in sterilization. Thresh believes that lime would furnish a satisfactory sterilization of effluents, but has not tried it. (Great Britain, 1902; A, Quest. 8917.)

3. ACIDS. — Most bacteria are much more sensitive to acids than to alkalies. This applies particularly to the typhoid and cholera germs. Rideal considers it feasible to use acids as germicides. He states that Stutzer found .05 per cent. of sulphuric acid fatal to cholera in fifteen minutes and .02 per cent. fatal in twenty-four hours; that Ivanoff found that .04 per cent. to .08 per cent. destroyed the cholera germs in the sewage of Berlin and of Potsdam; and that Kitasato found .08 per cent. sulphuric acid fatal to typhoid in fifteen minutes, a result

obtained also by Rideal himself. To furnish one million gallons of sewage with .08 per cent. sulphuric acid would require 6,640 pounds of acid, at an approximate cost of $332. Smaller amounts of acid might be used, since it would not be necessary to kill typhoid germs in so short a time as fifteen minutes; but, on the other hand, there is in most sewage a considerable amount of free alkali requiring neutralization before any effect of the acid would be obtained. On the whole, this process would seem to be an impractical one except in cases of emergency. It is interesting to note that the sewage of Worcester, Massachusetts, contains normally an average of .01 per cent. free sulphuric acid, or half enough to kill cholera germs in twenty-four hours.

4. OZONE. — Ozone has been used with more or less success in various parts of Germany, particularly at Weisbaden, for the sterilization of drinking water. Although the process has been most favorably spoken of by those in immediate care of the investigations, it has not generally been regarded as a success, and its use at Weisbaden has been discontinued. There can be no doubt as to the possibility of procuring a satisfactory effluent by this process when the water in question is a highly polluted river water. Whether the process would deal satisfactorily with a sewage effluent of considerable turbidity has not been determined. Rideal points out that ozone is but sparingly soluble. Such being the case, it might altogether fail to penetrate the solid masses in the effluent, since the rate at which a dissolved gas will thus penetrate solids is a direct function of its solubility.

The principal cause of the failure of the ozone process, however, seems to be its expense. If this is the case in water works it seems hardly possible that the process could be applied to sewage effluents as an additional safeguard after purification. There are no data at hand for estimating the cost of ozone treatment when applied to effluents. Besides the cost of treatment there is the very considerable cost of installation of the necessary machinery and towers.

5. CHLORINE AND ITS COMPOUNDS. — Chlorine is well known as a powerful germicide. It has long been known as a bleaching agent acting upon organic coloring matter, not directly, but by means of the free nascent oxygen which it liberates from the water in which it is dissolved. It is probable that its germicidal effect is similar. In other words, chlorine and ozone come to the same thing in the end, namely, nascent oxygen. Chlorine, however, has the advantage of

more ready solubility and of cheapness. It is used technically in three forms, all of which, however, act in the form of free chlorine, and not as compounds.

(*a*) *Chlorine Gas.* — Until within recent years chlorine has been manufactured commercially by the Weldon or some similar process. In the Weldon process hydrochloric acid is made to react with a complex mixture of manganese hydroxide and lime — "Weldon mud" — the reaction being essentially

$$MnO_2 + 4HCl = MnCl_2 + 2H_2O + Cl_2,$$

although in reality it is much more complex.

Recently, however, electrolytic processes have been developed by which the cost of manufacture has been materially reduced, particularly where cheap water power is available. The difficulties in the way of using gaseous chlorine are chiefly the cost of transportation, difficulty and danger in handling the gas, and the difficulty of accurately measuring the amount of gas added to the effluent. On the other hand, if the magnitude of the work would warrant the installation of an electric chlorine generating plant, there is reason to believe that there would be a great economy in the preparation and direct use of the gas, and the disadvantages noted would be obviated to a large extent. This matter of cost will be more fully discussed in another place.

(*b*) *Oxychloride Process. Hermite Process.* — The electrolysis of ordinary salt yields chlorine and caustic soda. It is claimed that if magnesium salts be present, as in sea water, a secondary reaction occurs by which there are produced certain oxides of chlorine. Three oxides of chlorine are known: Cl_2O and Cl_2O_3, the anhydrides of hypochlorous and chlorous acids, respectively, and chlorine peroxide, ClO_2. The latter is formed only under exceptional conditions and is extremely unstable. The others unite instantly with water to form the corresponding acids. There seems to be no legitimate reason for assuming the presence of any compounds in this electrolyzed sea water except compounds analogous to those which we know are produced in the case of the sodium and the calcium salts; namely, hypochlorites. The hypochlorite of magnesium seems to be an unstable compound, readily hydrolyzing to magnesium hydrate and hypochlorous acid. This fact is sufficient to account for the higher disinfectant value observed in the use of electrolyzed sea water as compared with equivalent amounts of available chlorine in the form of bleaching powder.

Rideal made an extensive series of experiments at Guildford, England, where he was able to test the effect of the "oxychloride" treatment upon raw and septic sewages and upon effluents of primary, secondary, and tertiary contact beds. The following tabulated summary of his results will show about what may be accomplished. By the term "available chlorine" (av. cl.) is meant the chlorine value as obtained by titration with arsenious acid. The results, especially as regards the removal of *B. coli*, are all that can be desired. Examinations of the few organisms remaining in the sewages and effluents after treatment showed them to be largely organisms of the hay bacillus group, aërobic spore-forming bacteria, which are probably beneficial in the further oxidation of the organic matter. Absolute sterilization required very high concentration of chlorine.

TABLE I — SUMMARY OF RIDEAL'S EXPERIMENTS AT GUILDFORD, ENGLAND, ON THE USE OF OXYCHLORIDE IN STERILIZING RAW SEWAGE, SEPTIC EFFLUENTS, AND THE EFFLUENTS OF PRIMARY, SECONDARY, AND TERTIARY CONTACT FILTERS

Sample of	Av. cl. Parts per million.	Time of contact.	ORGANISMS PER CUBIC CENTIMETER.					
			Total.		B. coli.		B. enteritidis.	
			Initial.	Final.	Initial.	Final less than	Initial.	Final less than
Sewage	30	4.3 hours	23,000,000	50,000	1,000,000	1.0	1,000	10.0
	50	4.3 hours	23,000,000	20	1,000,000	0.2	1,000	0.2
	70	4.3 hours	23,000,000	10	1,000,000	0.2	1,000	0.2
Septic effluent .	25 to 44	1.0 hour to 4.0 hours	2,500,000 to 4,500,000	20 to 600	100,000 to 1,000,000	1.0 to 0.2	10 to 1,000	1.0 to 0.2
First contact . .	20	40 minutes	. .	. .	100,000	0.2	. .	. .
	20	2 hours	. .	. .	. .	. .	20 to 100	0.2
Second contact .	10.6	. .	1,000,000 to 2,000,000	40	1,000,000	0.2	10 to 1,000	0.2
Third contact .	2.5	1.0 hour	. .	. .	1,000 to 10,000	0.2	. .	. .
	2.5	4.5 hours	. .	. .	. .	. .	10 to 100	1.0
	0.5	0.5 hour	. .	. .	1,000 to 10,000	0.2	. .	. .
	0.5	4.5 hours	. .	. .	. .	. .	10 to 100	0.2

(*c*) *Chloride of Lime.* — Chloride of lime, so called, is a mixture of calcium hypochlorite, calcium chloride, and impurities. Its efficiency as a disinfectant is due to the action of the hypochlorite, which decomposes slowly, giving off oxygen and free chlorine. Thresh has used chloride of lime at Ilford, England, for the sterilization of the effluent of a chemical precipitation plant. The addition of 140 to 200 parts of the chloride was found to give a nearly sterile effluent after the latter had flowed in a closed conduit for a mile. Commercial chloride in this country will average 35 per cent. available chlorine. Upon this basis the amount of chlorine used was 50 to 100 parts. The chloride would, therefore, seem to be as efficient as the so-called oxychloride, and in fact is probably a similar compound.

During the past year experiments have been carried on at this station to determine the efficiency of bleaching powder in the sterilization of the effluent of a trickling filter. A preliminary series of tests was first made, covering a wide range of concentrations of bleaching powder. These tests were made in bottles. A strong suspension of bleaching powder was made up and analyzed for available chlorine by the arsenious acid method. Portions of this solution were then measured into bottles, each containing 1 liter of the effluent from the trickling filter. The available chlorine was again determined in a sample withdrawn from the bottle. Determinations of the total organisms present in the effluent before adding the disinfectant and at stated intervals thereafter were made. Plates were made with standard nutrient gelatin, incubated at 20° and counted on the third day.

The results of these tests are summarized in Table II, and in Table III are calculated into percentage figures:

TABLE II — Preliminary Tests of Efficiency of Bleaching Powder in the Sterilization of a Trickling Filter Effluent

Temperature 20°

Experiment No.	Bottle No.	Av. cl. Parts per million.	Bacteria per Cubic Centimeter.					
			Initial.	Thirty minutes.	One hour.	Two hours.	Five hours.	Twenty hours.
1906								
I. Started February 13 . .	Blank	0.0	80,000	. .	. .	. .	. .	640,000
	1	3.5	70,000	850	. .	500	50	50
	2	10.6	80,000	500	. .	900	290	50
	3	55.0	90,000	50	. .	180	. .	50
	4	108.0	60,000	. .	. .	. .	. .	50
II. Started February 20 . .	Blank	0.0	133,000	. .	. .	. .	. .	. .
	1	25.0	122,000	190	90	. .	. .	. .
	2	25.0	99,000	100	180	. .	. .	. .
	3	37.0	128,000	50	80	. .	. .	. .
	4	100.0	124,000	27	36	. .	. .	. .
III. Started March 7	Blank	0.0	176,000	. .	. .	272,000	. .	. .
	1	2.0	153,000	. .	300	100	. .	. .
	2	5.0	130,000	. .	100	50	. .	. .
	3	5.0	146,000	. .	100	300	. .	. .
IV. Started March 13	Blank	0.0	80,000	. .	. .	86,000	. .	. .
	1	0.25	96,000	. .	33,000	38,000	. .	. .
	2	0.50	84,000	. .	9,100	4,600	. .	. .
V. Started March 20	Blank	0.0	230,000	. .	. .	200,000	. .	. .
	1	1.0	212,000	. .	39,000	1,900	. .	. .
	2	1.5	146,000	. .	500	100	. .	. .
VI. Started April 17	Blank	0.0	24,000	. .	. .	68,000	. .	. .
	1	5.0	25,000	. .	. .	500	. .	. .
	2	10.0	35,000	. .	. .	100	. .	. .

TABLE III — Summary of Preliminary Experiments

Experiment No.	Available chlorine. Parts per million.	Organisms Killed. Per Cent. of Initial Numbers.				
		Thirty minutes.	One hour.	Two hours.	Five hours.	Twenty hours.
I	3.5	98.7	. .	97.8	99.6	99.9
	10.6	98.9	. .	98.9	99.7	99.8
	55.0	99.9	. .	99.9	99.9	99.9
II	25.0	. .	99.8	99.9	. .	. .
	37.0	. .	99.94	. .	. .	. .
	100.0	. .	99.97	. .	. .	. .
III	2.0	. .	99.8	99.9	. .	. .
	5.0	. .	99.85	99.9	. .	. .
IV	0.25	. .	65.6	70.9	. .	. .
	0.50	. .	89.2	94.2	. .	. .
V	1.0	. .	81.5	99.1	. .	. .
	1.5	. .	99.6	99.9	. .	. .
VI	5.0	. .	98.2	98.2	. .	. .
	10.0	. .	. .	99.6	. .	. .

These results indicate clearly the practical impossibility of obtaining a sterile effluent in this way. Neither the addition of large amounts of bleaching powder, up to 100 parts of available chlorine per million, nor the storage of the effluent for periods of time up to twenty hours proved entirely effectual. It is further shown that a concentration of about five parts per million of available chlorine and a storage period of about two hours give practically the maximum efficiency possible with the process.

Upon this basis, therefore, a practical test of the process was made. A large trickling filter at the Sewage Experiment Station, filtering raw sewage at the rate of five thousand gallons per day, was used. A solution of bleaching powder was made up in a fifty-gallon barrel of such strength that when added to the effluent at the rate of two gallons per hour it would supply the effluent with about five parts per million of available chlorine. It was not possible on such a small scale to regulate the flow accurately, and there was some variation in the amount of available chlorine found in the effluent. This value was determined each day when the samples were taken for bacterial examination. It averaged about five parts per million, and the variation was no more than may be expected in the case of a larger plant with less careful supervision.

The mixed effluent and sterilizing solution passed into a settling basin built on the principle of a Dortmund tank, and designed to give a mean storage of approximately two hours. Regular examinations of the effluent before admixture with the sterilizing solution and after this two-hour period of contact were made. The total bacteria growing upon gelatin at 20° were determined by the usual plating method.

The removal of total organisms was found to be 99.96 per cent. The determination of *B. coli* was made by the bile broth presumptive test recommended by Jackson (1906). Positive results were obtained in 33 per cent. of the tests made upon the untreated effluent at a dilution of $\frac{1}{1000000}$, indicating that the organism was present on the average to the extent of about 330,000 per cubic centimeter. After treatment positive tests were obtained in 22 per cent. of 1 c.c. samples, indicating that on the average there was one organism present in 5 c.c. of effluent. Upon this basis the average removal of fermenting organisms (presumably *B. coli*) was 99.993 per cent.

The detailed results of this series are shown in Table IV.

TABLE IV—EXAMINATIONS OF EFFLUENT FROM TRICKLING FILTER BEFORE AND AFTER STERILIZATION WITH CHLORIDE OF LIME, AVAILABLE CHLORINE, FIVE PARTS PER MILLION

Date.	ORGANISMS PER CUBIC CENTIMETER.		GAS PRODUCTION IN BILE BROTH.	
	Before treatment.	After treatment.	Before treatment. Dilution, $\frac{1}{1000000}$ c.c.	After treatment. Dilution, 1 c.c.
August, 1906.				
11	270,000	69	+ 0	+ 0
13	630,000	41	0 0	+ 0
14	135,000	406	+ +	+ 0
15	230,000	21	0 0	0 0
16	250,000	37	+ 0	0 0
18	110,000	40	0 0	+ 0
20	90,000	54	+ 0	0 0
21	220,000	22	0 0	0 0
23	. .	. .	+ 0	0 0
Average	240,000	86	33 per cent.	22 per cent.
Average removal		99.96 per cent.		99.993 per cent.

A series of laboratory experiments was carried on at the same time to determine the germicidal power of the bleaching powder solution upon *B. typhosus*. Strong bouillon cultures of this organism were made in sterile tap water. After the number of organisms initially present was determined, suitable amounts of the bleaching powder solution were added, and determinations were made of the organisms remaining alive at stated intervals. The results of these tests are shown in Table V.[1]

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[1] The writers desire to express their acknowledgments to Dr. R. P. Cowles, of Johns Hopkins University, for valuable assistance in carrying out the bacteriological work involved in these experiments.

TABLE V — EXPERIMENTAL RESULTS SHOWING THE GERMICIDAL ACTION OF BLEACHING POWDER SOLUTIONS UPON B. TYPHOSUS.

Experiment No.	Available chlorine. Parts per million.	ORGANISMS PER CUBIC CENTIMETER.							
		Initial.	One minute.	Twenty minutes.	Forty minutes.	Sixty minutes.	Two hours.	Four hours.	Twenty-four hours.
I	4.2	1,300,000	1,000,000	300,000	59,000	9,800	800	. .	15
II	5.4	5,500,000	4,450,000	1,350,000	850,000	675,000	450,000	450,000	6,000
III	4.7	1,200,000	. .	465,000	190,000	133,000	50,000	. .	54
IV	6.9	3,200,000	20,000	200	. .	300	20	. .	10
V	3.5	5,800,000	20,000	800	200	500	11	0	1
VI	3.5	3,200,000	70,000	200	100	200	26	16	8

Except in Experiment II, the reduction in twenty-four hours is practically complete, and in two hours is satisfactory for all purposes for which such treatment would be undertaken, the enormous initial numbers being taken into consideration. The phenomenon of a few specially resistant organisms is a very common one in all investigations of germicides, whether of heat, cold, desiccation, or chemical disinfection, and is particularly marked here. The results of Experiment II are in disagreement with the others. Whether they represent a specially resistant culture of the organisms or whether the results are due to some error in the experiment cannot be stated.

Summarizing, it may be said, as a result of these experiments, that a concentration of five parts per million of available chlorine from bleaching powder, acting for a period of two hours, will remove 99.96 per cent. of the total bacteria and practically eliminate the intestinal organism, *B. coli,* and any typhoid organism present.

COST OF CHLORINE IN VARIOUS FORMS

With the electrolytic process in use at Niagara Falls, where cheap power is available, the manufacture of chlorine and of bleaching powder is carried out very cheaply. Commercial bleaching powder can be purchased in the Eastern market at about 1 cent per pound, guaranteed 40 per cent. available chlorine. In other parts of the country the additional freight charges might make the cost somewhat higher. On the convenient basis of 1 cent the actual cost of the chlorine is 2.5 cents, and the cost of adding one part per million of available chlorine to the effluent would be 21 cents per million gallons, or $1.05 per million gallons for five parts. Bleaching powder is made by the action of chlorine upon caustic lime, so that chlorine in this form will obviously cost more than as the free gas. To develop at once its full bleaching or disinfecting action it is necessary to use it in acid solution, as is done in commercial bleaching processes. An investigation of the reactions involved leads us to believe that the total available chlorine, as indicated by the analysis, is eventually set free in a solution of organic matter, such as sewage, especially in the presence of sunlight; but it is probable that the reaction proceeds more and more slowly as it reaches the end, and that in the time available for treatment it is not safe to count upon more than 75 per cent. of the available chlorine being actually used. Rideal observed that there was an immediate loss of available chlorine corresponding roughly to the strength of the sewage or effluent under treatment, and that afterward the loss of available chlorine was very gradual, his explanation being that there were two distinct kinds of organic matter present. With the knowledge that free carbonic acid, always present in sewage, is able to react with the hypochlorite liberating chlorine, we are inclined to the view that the preliminary action noticed was due to this cause. Free gaseous chlorine, on the other hand, is all "available" and ready for immediate use. As the time element is one of immense importance in the design and cost of a sterilization plant, this item alone deserves much weight. Moreover, the use of chlorine gas possesses certain other advantages over the bleaching powder. It adds nothing else to the sewage, whereas bleaching powder adds a certain amount of insoluble material and considerable hardness. It seemed advisable, therefore, to obtain, if possible, some actual cost data on the production of electrolytic chlorine. Through the courtesy of Messrs. A. G. Paine, Jr., president, and J. R. Crocker, superintendent, of the New York and Pennsylvania Company, the following statistics have been obtained, based upon actual costs in a large paper mill using the McDonald electrolytic cell.

Each cell will produce 22 pounds of chlorine and 22 pounds of caustic soda per day of twenty-four hours, with the use of 2.08 kilowatts of current. One kilowatt hour is therefore equal to 0.46 pound of chlorine. The cost of power, made under the most favorable conditions by a modern compound condensing engine, burning "run-of-the-mine" coal at $2 per ton, is estimated at not over $52 per kilowatt year, or 0.6 cent per kilowatt hour. The cost of chlorine is therefore about 1.3 cents per pound, a trifle over half the cost of the chlorine in bleaching powder. If, as we have intimated, this form of chlorine shall be found to be 25 per cent. more efficient than the other, and if the original cost of installation be not too great, it seems that for large works, at least, the manufacture of chlorine might well be undertaken. Even at 2 cents per pound the cost of treating a million gallons of effluent with one part per million of chlorine would be reduced to 16.6 cents, or 83 cents per million gallons for five parts. Moreover, this expense would be considerably offset by the value of the caustic soda produced at the same time.

6. COPPER AND ITS COMPOUNDS. — Rideal has experimented recently with copper salts, and finds them efficient in the sterilization of effluents. While its expense would seem to prohibit the general use of copper, it is undoubtedly a suitable disinfectant for water cress and oyster beds.

Johnson and Copeland (1905) obtained the following reduction of total organisms in effluents:

	Copper sulphate. Rate per million.	PER CENT. REDUCTION.	
		Three hours.	Twenty-four hours.
First series, average of three sets	5	90.0	99.9
	10	98.0	99.95
	20	98.5	99.96
Second series, average of three sets	10	40.0	99.7
	20	60.0	99.9
	40	88.0	99.95

They find action most rapid during the first hour. They estimate the cost for chemicals alone at $5 per million gallons of effluent, treated with ten parts per million, or $10 for twenty parts, which they consider prohibitive. More experimental data on the use of copper salts are needed before we can judge of their value. Their effect will also depend largely upon the character of the water.

7. MISCELLANEOUS. (*a*) *Permanganate.* — Potassium and sodium permanganate have been used at times for the oxidation of organic matter in streams. At London sodium permanganate is added to the Thames regularly when it becomes too foul at times of low water. The application here is to destroy odors and putrescible material, but it

undoubtedly partially sterilizes the water. Its use has been proposed for rendering effluents of chemical precipitation plants non-putrescible. The cost is greater than chlorine compounds, which would seem to be fully as efficient.

(*b*) "*Amines*" *Process.*—This process was developed by H. Wollheim in England. It is claimed that trimethylamine treated with lime or other alkali produces a very poisonous substance. Herring brine is used for the amine. A large excess of lime is used, and the mixture used to precipitate crude sewage. Klein made a test of the process at West Horn in 1889, and found that a clear, non-putrescent, and sterile effluent could be obtained. Similar results were obtained at Wimbledon, where the bacteria on the sewage were reduced from 768,000 to none. The sludge is also non-putrescible. The process has never been developed further. (Great Britain, 1902, A, Quest. 5206.)

CONCLUSIONS

Summarizing the more important points of the present discussion, we may state:

Ordinarily it is not the function of the sewage filter to remove bacteria from the sewage. In the case of the modern rapid filters, we are not at all certain as to the fate of the dangerous germs of disease.

Where extensive shellfish interests are at stake it will sooner or later become necessary to protect the neighboring waters from pollution, and in this case the total absence of pathogenic bacteria must be assured.

This result may be attained by the ultimate use of sand filters or by chemical sterilization.

Among the possible agents for use in sterilization, chlorine, either as chloride of lime or as free gaseous chlorine, appears to have the highest cost efficiency. An amount of the former which will yield five parts per million of available chlorine was found to have destroyed 99.96 per cent. of the total bacteria and practically all of the *B. coli* present in the effluent of a trickling filter treating crude Boston sewage, the time of contact having been two hours. The estimated cost of such treatment is $1.05 per million gallons of sewage.

The use of gaseous chlorine, manufactured at the disposal works by electrolytic methods, would in the case of larger works considerably reduce this cost. An approximate estimate places the cost of such treatment at about 85 cents per million gallons of sewage, allowing five parts of chlorine per million. It is probable that the chlorine in this form is somewhat more efficient.

The process as thus outlined is entirely feasible, and in the case of large works it is cheaper than sand filtration. Further experiments upon a large scale are now under way to test the results of these experiments and to get more accurate cost and efficiency data.

Among other agents which may be used, copper sulphate seems to be the most promising, and further experimental data are needed to show what efficiency may be expected from its use.

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THE PREVENTION OF STREAM POLLUTION BY STRAWBOARD WASTE[1]

BY EARLE BERNARD PHELPS

INTRODUCTION

THE disposal of the waste liquors resulting from the manufacture of strawboard forms one of the most important problems connected with the prevention of stream pollution. In 1900 the total production of strawboard in the United States was 157,534 tons at 59 factories. For each ton of strawboard manufactured there were discharged into the streams about 65,000 gallons of waste liquor that contained 1,173 pounds of straw and mineral matter and 490 pounds of lime. The total waste discharged into the streams in 1900 amounted to 10,239,710,000 gallons of liquor, containing 184,777,382 pounds of straw and mineral matter and 77,191,660 pounds of lime. This enormous waste was discharged by 59 plants of various sizes, but as most of these mills are along small streams the resulting pollution is very apparent. Some idea of the amount of this pollution may be obtained from Plate I, *B*. The baled straw shown is a three months' supply for a mill having an output of 30 tons a day. In three months one-third of this straw will be discharged into the stream as waste matter.

The problem of the disposal of the waste water has been studied by Professor R. L. Sackett, of Earlham College,[2] and the present investigation is a continuation of the work of Sackett along new lines. In approaching the problem it is necessary to keep constantly in mind the economic conditions affecting the industry. The low selling price of the

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[1] Reprinted by permission from Water-Supply and Irrigation Paper, No. 189, United States Geological Survey.

[2] Water-Supply and Irrigation Paper, No. 113, United States Geological Survey, 1905, pp. 9–35.

PLATE I

A. VIEW OF STRAWBOARD MILL

B. PILES OF BALED STRAW
One-third, by weight, will be wasted in three months

product, the immense volume of waste water necessary, and the fact that competition between strawboard and other forms of cheap board is active, make it quite impracticable for the manufacturers to adopt any process involving serious expenditures. The solution of the problem must be sought in a process which will show an actual saving. Aside from the matter of the recovery of by-products of commercial value, this means that the process must compete in cost with the present unsatisfactory sedimentation methods employed at many of the mills, to the cost of which may reasonably be added the almost constant expense of lawsuits and injunction proceedings, the latter in some cases causing the closing of the mill.

As a result of this study it has been found that sedimentation of the waste water for a short period, followed by filtration thru sand at high rates—the so-called mechanical filtration, but without coagulants—will produce an effluent from which more than 90 per cent. of the suspended organic matter has been removed. This effluent, while not ideal, is probably purified to such an extent that, allowing a factor of safety of 2 for impurities in the stream, it may be discharged into streams having a minimum flow of twice the discharge of the mill without producing a nuisance. The cost of filtration by this process, after the initial cost of installation, is practically reduced to the cost of pumping the wash water. The final result is a heavy sludge, whose volume is less than 5 per cent. of the initial volume of the waste. Ultimate disposal of this sludge could be made either by discharge over land for spontaneous evaporation, by evaporation by means of artificial heat, or by filter pressing. The sludge has some value as a fertilizer, and there are also certain commercial uses to which it seems to be adapted.

THE MANUFACTURE OF STRAWBOARD

The following description of the manufacture of strawboard is taken from Professor Sackett's paper:[1]

Steaming in Rotaries

"The straw is first subjected to a cooking process by steam and lime. A large ellipsoidal rotating steel boiler called a 'rotary' . . . is filled with straw, which is then cooked down with steam, then again filled and cooked down until the rotary boiler has been completely filled. The process of filling a rotary occupies from six to twelve hours. The final charge consists of about 6 tons of straw and 30 bushels, or 2,100 pounds, of lime in the form of milk. This mixture is then rotated and cooked under 40 pounds of steam pressure for twelve hours. . . . This apparently severe chemical and mechanical action results in a rapid softening of the woody fiber and in the reduction of the straw to a dark yellow, pulpy mass. This 'stock,' as it is called, is stacked in piles 10 to 15 feet high to drain. Concerning the action which takes place in the rotaries, the *Journal of the Society of Chemical Industry*[2] says:

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[1] Water-Supply and Irrigation Paper, No. 113, United States Geological Survey, 1905, pp. 13–15.

"'The chemical action of the milk of lime on the incrusting materials surrounding the straw fiber is not a vigorous one. These incrusting materials are not completely, nor, indeed, to a great extent, separated from the cellulose. The mineral matter remains in the product practically untouched, and if any less quantity than that corresponding to the percentage in the original straw operated upon exists in the prepared pulp, it is due rather to the washing after digestion than to any solvent action of the milk of lime. Milk of lime under certain conditions has a bleaching action upon the straw. It neutralizes the organic acids usually found when fibrous plants are heated for any length of time in the presence of water.'

"The yield of pulp at this point will be from 75 to 80 per cent. of the weight of the original material. . . .

"The material is allowed to stand in these piles for twenty-four hours or more to drain. After it is thus drained it contains about 50 per cent. of water and from 6 to 7 per cent. of lime. This is equivalent to from 12 to 14 per cent. of lime in the dry straw. Since the original charge of lime and straw was in the proportion of 2,100 pounds of lime to 14,100 pounds of straw and lime, or about 14 per cent. lime, practically none of the latter has drained out with the condensed steam. This drainage from the stock piles forms but a small part of the waste sewage. It is straw colored and very turbid, carrying a small quantity of fiber broken fine in the rotaries.

WASHING

"This process is much more drastic, and it is here that the great volume of waste is produced.

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[2] For February 28, 1894, p. 101.

"The stock is run thru washing machines for the purpose of removing the lime. . . . The washing machine consists of an oval channel about 3 feet wide, around which the stock travels, being supplied with copious volumes of water. Across this channel is placed a cylinder, 42 inches in diameter and 42 inches long, having longitudinal ribs or flanges about three-fourths of an inch square in section and three-fourths of an inch apart. Meshing with this, like the teeth of geared wheels, is an idler, below, of similar size and form. These wheels, revolving, lift the water and straw to a level several inches higher than that in the oval channel, whence it flows by gravity halfway around its course to a point where it meets a revolving brass screen of fine mesh, thru which a part of the water escapes, carrying with it the finer particles of fiber and free lime. The remaining straw, with additional volumes of fresh water, now passes many times thru the rolls, which further mash and break the fiber, and around to the screen, where more straw, lime, and water escape. . . .

DRYING

"After the washing process the straw, with a considerable volume of water, is led to a train of rolls, consisting of three parts—first, the wet end; second, the hot rolls; third, the trimming and cutting machine. As it comes from the washers the material is run into vats, where it is mixt with large quantities of water and past over hollow cylinders having fine wire-cloth faces, which allow the water to escape, leaving the fiber on the surface of the cylinder. The fiber is then taken by woolen felts, which are prest down on the surface of the cylinder. This makes a web of paper on the felt. The pulp, which is now about one-third straw and two-thirds water, travels up and down, over and under a double train of hot rolls, heated by steam that is carried in thru hollow bearings. As the pulp passes on thru the train it is constantly prest and dried, until finally it is separated from its cloth support and goes to the trimming machine, where it is cut into sheets of proper size. It now contains about 10 per cent. of water and a small quantity of lime. The board is manufactured in many thicknesses and weights. Just before it is trimmed it may be coated on one or both sides with a thin paper facing or finish."

THE DISPOSAL OF WASTE LIQUOR

METHOD IN USE

At present, so far as can be learned, the only method of treating the waste liquor in actual use by the manufacturers is that of prolonged sedimentation in open fields. The overflow water from these fields is not satisfactorily clarified at all times of the year. During cold weather it is fairly clear and can be discharged without serious harm. During the summer, however, a very offensive fermentation takes place in the accumulated sludge, giving rise to objectionable odors and blackening the water. At such times the aim is to hold all the waste water in the ponds, trusting to evaporation and to the storage capacity of the ponds until cold weather again relieves the situation and the increased flow of the stream permits a discharge of the surplus water stored. The most serious objection to this process, which seems to be satisfactory to riparian owners below, is the fact that these ponds are continually filling up with a soft mud, which destroys their function as sedimentation basins and renders them useless for other purposes. The time must come, even in the most favorable situation, when additional land cannot be afforded for this purpose. A second objection is that the accumulated sludge is held for years and slowly putrefies. The organic matter of the waste is but slightly nitrogenous and is fairly stable. It probably contains some reducing sugars capable of rapid fermentation if kept in concentrated form and if given at the same time some nitrogenous matter. The slow decomposition of the stable straw material supplies the necessary nitrogen, and the result is a very rapid multiplication of the putrefactive organisms and an accompanying offensive putrefaction. Laboratory experiments have shown that the soluble sugars by themselves, as found in a filtered sample of the waste water, do not ferment or decompose, and that even the unfiltered waste is not subject to a quick putrefaction such as characterizes sewage matter. In fact, bottles have been stored for over a year without any evidence of fermentation, and it may very reasonably be held that the necessary nitrogen comes largely from the bottom of the ponds, rather than from the sludge itself. Inspection of the stream below one of these ponds showed that, while putrefaction was barely noticeable and not at all offensive in the pond itself, the admixture of some of this water with that of a small brook, highly polluted with the refuse of a slaughterhouse, had immediately a very offensive result. Investigation of the condition of the streams below these mills reveals the fact that the most serious nuisance generally occurs at a point very remote from the mill itself. It is directly caused by the slow decomposition of material deposited in the bed of the stream. The conditions are doubtless aggravated by the simultaneous precipitation of other polluting substances, notably sewage, brought about by the free lime of the strawboard waste.

The effective solution of the problem lies in the rapid separation of the solid matter from the dissolved sugars and lime and a subsequent dilution of the latter so that they shall be innocuous. The insoluble material when dry is entirely stable. The present system, therefore, is not only unsatisfactory from the economic side, but it works on a wrong principle for satisfactory purification in that it encourages putrefaction.

The cost of these ponds at four of the larger mills of the United Box-board and Paper Company, as stated by an official of that company, has been, in round numbers, to date, \$10,000, \$15,000, \$18,000, and \$8,000. In the last case the results were still unsatisfactory, and after a considerable expenditure on other experiments the company has been obliged to close this mill.

PROPOSED METHODS

Centrifugal Separation

Centrifugal separation of the solid matter from the liquor was tried at Tiffin, Ohio, in 1904. It has not been possible to obtain more definite information concerning those experiments than is contained in the trade papers of that time. It is understood, however, that the results were quite satisfactory, but that the cost of operation was prohibitive. The account referred to (see *Paper Trade Journal,* February 2, 1905) states that the effluent was clear and colorless and that the residue was found by analysis to be valuable as a fertilizer.

Precipitation with Carbon Dioxide

As a result of his investigations Sackett concluded that plain sedimentation would remove about 10 per cent. of the suspended solids in twenty-four hours, and that during the ensuing twenty-four hours the additional removal was very slight; that good purification could be effected by chemical precipitation only by using sulfate of aluminum in excessive amounts and at an entirely prohibitive cost; that sulfate of iron gave inferior and unsatisfactory results; and, finally, that by a process of precipitation with carbon dioxide and lime an average removal of 85 per cent. of the suspended matter could be effected.

The process recommended by Sackett consists in (*a*) saturation of the waste water with carbon dioxide to form bicarbonate with all the lime present; (*b*) addition of lime equal to that already present to form a normal carbonate; (*c*) sedimentation of the precipitated carbonate and consequent dragging down of some of the organic matter; and (*d*) filtration of the resulting clarified liquor thru sand at rates of 500,000 to 2,000,000 gallons per acre per day. This process has never been used by manufacturers so far as the writer is aware.

It does not appear from Sackett's report that he tried the simpler but equivalent process of saturating one-half the total volume with carbon dioxide and then mixing with the other half. Such treatment would doubtless have precipitated the calcium fully as well, with a saving of one-half the carbon dioxide and all the lime required by the process recommended. An examination of the analytical data shows that the removal of suspended organic matter accomplished by the precipitation and sedimentation processes varied from zero to 75 per cent., with a mean value of 37 per cent. Experiments on plain sedimentation, made by the writer both in the laboratory and on a large scale in the field, show it to be equally efficient for the removal of suspended organic matter. The fact is that the precipitated carbonate of calcium is very finely divided and has no value as a coagulant. The process is valuable to remove the lime, if that be desirable. It hardly seems, however, that the value of discharging a lime-free effluent into the hard waters of the Middle West is at all commensurate with the cost of treatment.

LABORATORY INVESTIGATIONS

Chemical Study of the Soluble Matter

Several barrels of the strawboard waste liquor were shipped to Boston from one of the mills of the United Box-board and Paper Company, and on this material a series of investigations was carried out. Sackett's experiments with various coagulants were repeated and his results confirmed. It was found that the poor results obtained with any but excessive amounts of iron or aluminum sulfate were due to the presence of a substance in solution which forms soluble combinations with those metals. This substance was isolated by precipitation with hydrochloric acid, and after repeated purification was found to have the following properties: It is an organic acid, probably related to the humic or ulmic acids of the soil and peaty waters. It exists in the waste water as a calcium salt, having partially neutralized the free lime used in the digestion. It is precipitated by strong acids, coming out as a dark-red substance which turns brown on slow oxidation in the air. It can be redissolved in a solution of sodium carbonate, giving a garnet-red solution. It dissolves in alcohol, giving a deep-red color, but oxidizes to a brown, insoluble substance. Its calcium and aluminum salts are not soluble when prepared from the insoluble acid, but the acid cannot be precipitated from its solutions by these metals. It appears that there are formed colloidal suspensions of the calcium and aluminum salts. The solutions have a bright yellow color and colloidal appearance. The iron compound is similar but of a green color. The sodium salt appears to be soluble and to dissociate into deeply colored acid ions.

A second substance isolated from the waste has the following properties: It is soluble in water and not precipitated by any of the common precipitants, including alum, ferrous sulfate, lead acetate (normal or basic), carbonates, hydrates, or acids. It is responsible for a large part of the color of the water and cannot be removed by any practical method.

These two substances with the lime make up the greater part of the dissolved material. They are entirely stable and do not undergo putrefactive change when stored in the incubator, either alone or mixt with varying proportions of tap water. In addition to these substances small amounts of reducible sugars are present. The amount is so small that in any material dilution of the liquor the dilution of these sugars will prevent their manifesting themselves as putrefactive agents. These sugars are probably responsible for the slow fermentation in the sedimentation fields after the lime has been sufficiently removed to allow the action to proceed.

PRECIPITATION EXPERIMENTS

Substances in suspension may be removed by filtration or by sedimentation. When the material is finely divided or of low specific gravity, either process may sometimes be assisted by the application of suitable coagulants. These tend to clot the small particles into larger flakes, and by their own greater density carry the clotted material to the bottom. Two chemical substances are required, as a rule, to produce the necessary precipitation, one of which is generally the calcium bicarbonate found normally in the water. This calcium salt reacts with the sulfates of certain metals, notably iron or aluminum, to produce the sulfate of calcium and the hydrates of the corresponding metals. In the present case there is a large amount of free lime in solution. It would seem, therefore, that precipitation might be brought about by the addition of either copperas or alum. Experiments showed that copperas failed to react at all and that fair results could be obtained with alum, but only at a prohibitive cost. As will be seen in the accompanying tables of results, the removal of organic matter by alum was much better than that obtained by plain sedimentation.

RESULTS OF PRECIPITATION EXPERIMENT ON STRAWBOARD WASTE

Sample.	Suspended Solids.	
	Present.	Removed.
	Parts per million.	Per cent.
Raw waste	1,350	. .
Waste after two hours' sedimentation:		
With coagulant *	120	91
Without coagulant	560	59

* The coagulant used was alum in the proportion of 10 grains of sulfate of aluminum per gallon.

Increasing the amount of aluminum sulfate gave practically the same result. Smaller amounts gave indefinite and at times negative results. The use of 10 grains per gallon of sulfate of aluminum at 1 cent a pound would represent a cost of about 43 cents per ton of finished product of the mill. The studies on the nature of the dissolved material already referred to explain these poor precipitation results with alum. Soluble compounds are formed with the organic acid, instead of aluminum hydrate, the desired coagulant.

Experiments were made with carbon dioxide according to Sackett's procedure, and his results were confirmed. There was but a slight removal of suspended organic matter—about 30 per cent.—which could be produced equally well by plain sedimentation. In fact, under actual working conditions sedimentation was found to be even more effective.

SEDIMENTATION EXPERIMENTS

In a study of plain sedimentation it is important to know the rate of sedimentation and how this varies with the time. With satisfactory data on this point it is possible to estimate the amount of clarification which may be expected to result from any given period of sedimentation, and to establish the most suitable period for the case in hand. It was observed early in this work that apparently a large portion of the suspended solids settled out in a very short time, and that to all appearances further improvement was extremely slow. The determinations of the total solids given in the following table show that there is no practical advantage in sedimentation for a period of much over one hour.

EFFECT OF SEDIMENTATION ON STRAWBOARD WASTE

	Suspended Solids.	
	Present.	Removed.
	Parts per million.	Per cent.
Liquor at beginning of experiment	1,344	. .
Supernatant after one hour	548	59
Supernatant after twelve hours	494	63

SUMMARY

As a summary of the laboratory investigations, it may be said that the removal of the dissolved coloring and other organic matter and the lime in an economical way is impractical; that by plain sedimentation a removal of suspended material practically as good as the best removal observed with the use of a coagulant can be accomplished; and that a sedimentation period of one hour is sufficient for the removal of about 60 per cent. of the suspended solids, a longer period increasing this removal but slightly. For the removal of the remainder of the suspended matter there remains but one process—filtration. It was therefore decided to make an actual field test of this method of purification. The very large amounts of suspended matter to be handled seemed to eliminate at the outset the possible use of slow sand filtration, so that it was obvious that mechanical filtration at high rates and with some provision for removing the accumulated sludge from the surface of the filters was the only system applicable to the problem.

FIELD INVESTIGATIONS

ACKNOWLEDGMENTS

In March and April, 1905, a visit was made to one of the mills of the United Box-board and Paper Company, at Urbana, Ohio. Every opportunity was given the writer to make a thoro study of the processes of manufacture, and it is here desired to make suitable acknowledgments to Mr. Rush E. Evans, division manager, and to Mr. D. C. Amerine, superintendent, for the many courtesies extended, both on that occasion and during the subsequent visit for the installation of an experimental plant.

AMOUNT OF WATER USED

It was found that conditions were favorable for measuring the entire volume of waste water, as it left the mill in a waste flume. As a check on these measurements, observations of the rate of operation of the pump were made, all the water used being pumped from a large well under the mill.

A rectangular flume, whose mean width was 1.96 feet, was used for these measurements. A distance of 100 feet was measured off in a portion where the slope was practically constant. To determine the velocity of flow at various elevations in the flume, a float was used which was nearly as long as the width of the flume and was provided with vanes underneath, so that it always floated squarely across the flume. The following observations are the mean of a large number made at various elevations of water in the flume.

MEAN DEPTH, VELOCITY, AND FLOW OF WASTE WATER IN FLUME OF MILL OF UNITED BOX-BOARD AND PAPER COMPANY, URBANA, OHIO

Depth of water.	Velocity of flow.	Area of cross-section.	Flow.
Foot.	Feet per second.	Square foot.	Cubic feet per second.
.19	3.1	.37	1.1
.25	3.8	.49	1.9
.29	4.0	.57	2.3
.31	4.2	.61	2.6
.33	4.3	.65	2.8
.37	4.5	.72	3.2

These values of the depth and the corresponding flow were then plotted. The most probable line was drawn thru them, and that curve was used as the true rating curve of the flume in the subsequent calculations.

During the two hours from 10.45 A.M. to 12.45 P.M., March 30, a test run was made to determine the total flow. The following readings of the elevation of the water in the waste flume were made, and from these readings has been calculated the total flow for the two hours.

FLOW OF WASTE WATER IN FLUME OF MILL OF UNITED BOX-BOARD AND PAPER COMPANY, URBANA, OHIO, MARCH 30, 1905

Time.	Depth.	Flow.	Flow for Fifteen-Minute Period.	
			Mean rate of flow.	Total flow.
	Foot.	Cubic feet per second.	Cubic feet per second.	Cubic feet.
10.45 A.M.	.29	2.3	. .	. .
11.00 A.M.	.29	2.3	2.3	2,100
11.15 A.M.	.33	2.8	2.6	2,300
11.30 A.M.	.33	2.8	2.8	2,500
11.45 A.M.	.33	2.8	2.8	2,500
12.00 M.	.33	2.8	2.8	2,500
12.15 P.M.	.38	3.4	3.1	2,800
12.30 P.M.	.36	3.1	3.2	3,000
12.45 P.M.	.33	2.8	3.0	2,700
Total	. .	. .	. .	20,400

This total flow of 20,400 cubic feet in two hours represents an average flow of 2.69 cubic feet per second.

A few measurements were made of the water running in the waste flume when the beaters were not running and again when the paper machine was stopt. The flow from the machine was found to be fairly constant at 1.4 cubic feet per second, and the beaters were found to discharge amounts of 1.2 to 1.8 cubic feet per second, depending on the number of beaters in operation at the time.

During the test run observations of the rate of operation of the pump were made at short intervals. The pump was a double-acting duplex Smith-Vaile pump, with water cylinders 14 inches in diameter and with a stroke of 15 inches. Slip being neglected, the capacity of the pump is, therefore, 1.335 cubic feet per stroke, or 5.340 cubic feet per complete revolution. Following are the observations made on the pump and the calculated rate of pumping:

DISCHARGE OF PUMP AT MILL OF UNITED BOX-BOARD AND PAPER COMPANY, URBANA, OHIO, MARCH 30, 1905

Time.	Revolutions per minute.	Discharge.	Time between observations.	DISCHARGE OF PUMP BETWEEN OBSERVATIONS.	
				Mean.	Total.
		Cubic feet per second.	Seconds.	Cubic feet per second.	Cubic feet.
10.45 A.M.	31	2.28	. .	. .	. .
11.00 A.M.	30	2.67	900	2.43	2,457
11.15 A.M.	30	2.67	900	2.67	2,403
11.30 A.M.	30	2.67	900	2.67	2,403
11.45 A.M.	30	2.67	900	2.67	2,403
11.55 A.M.	30	2.67	600	2.67	1,602
11.55 A.M.	40*	3.58	. .	. .	. .
12.25 P.M.	40	3.58	1,800	3.58	6,444
12.45 P.M.	38	3.57	1,200	3.48	4,176
Total pumped	. .	. .	. .	. .	21,888

* Changed.

During the test four of the seven rotaries were filled. Two hundred and fifty cubic feet of water are required for each rotary, or 1,750 cubic feet for the seven. Since all seven are filled only once in twenty-four hours, the water used in this process amounts to 73 cubic feet per hour for the whole day. The water so used during the two hours of the test was, however, 750 cubic feet, an excess of 604 cubic feet over the normal mean consumption. The amount pumped must, therefore, be corrected by this amount. The elevation of the water in the supply tank was 1 foot greater at the end of the test than at the start. The diameter of the tank is 12 feet. This storage amounts, therefore, to 113 cubic feet, and must also be applied as a correction. A small stream of water was being wasted from the engine condenser. This stream was roughly estimated by float measurements to be about 0.3 cubic foot per second, or 2,160 cubic feet during the two hours of the run. These three corrections amount to 2,888 cubic feet, which is to be subtracted from the total pumped, 21,888 cubic feet, leaving a net pumpage of 19,000 cubic feet for two hours, or 2.64 cubic feet per second, a result in close agreement with the 2.69 cubic feet per second measured in the waste flume.

RAW MATERIAL AND FINISHED PRODUCT

During this test run, careful weighings were made of the amount of finished board produced. The figures were as follows:

PRODUCT OF MILL OF UNITED BOX-BOARD AND PAPER COMPANY, URBANA, OHIO, FOR TWO HOURS, MARCH 30, 1905

	Pounds
Gross product	4,841
Waste and trimmings	196
Net product	4,645

Dividing the total amount of waste liquor, 20,400 cubic feet, by the amount of net product gives 8,780 cubic feet, or, roughly, 65,000 gallons per ton of product. As the daily product of this mill is about 30 tons, about 1,900,000 gallons of waste water are discharged each day.

The charge of one rotary was carefully weighed. There were used 133 bales of straw, of an average weight of 102 pounds, or a total of 13,600 pounds of straw. The amount of lime was weighed, but the weight was somewhat in excess of the average amount used, as determined by the total weekly consumption of lime. The latter figure, which is 2,100 pounds per charge, will therefore be used. The total amount of straw used in the 7 rotaries is accordingly 95,200 pounds per day, and of lime, 14,700 pounds per day. Subtracting from this the 60,000 pounds of finished product leaves a waste of 35,200 pounds of vegetable and mineral matter from the straw and 14,700 pounds of lime, which is carried off each day in the 1.9 million gallons of water. Calculated into parts per million, these figures give about 2,400 parts of straw material and about 1,000 parts of lime. Analyses of average samples of the waste water gave about 600 parts of calcium or 840 parts of lime per million. The difference is due partly to the loss of lime during the draining of the digested pulp and largely to impurity of the lime.

CHARACTER OF THE MACHINE WATER

Determinations of the lime were made on the well water and on the various waste waters. It was found that the greater part of the lime is washt out in the beaters, and that the water from the machine contained but little lime in excess of that in the raw water. The actual figures were:

CALCIUM IN WASTE WATER

(Parts per million)

Well water	238
Beater water	1,164
Machine water	380

A rough determination of the impurities was made by precipitating the lime and organic matter, with a large excess of alum. A 6-inch column of the beater water gave a layer of precipitate an inch thick, while the amount obtained from the machine water was too small to be measured.

MECHANICAL FILTRATION

In May, 1906, a small experimental filter was installed at this mill, in order to obtain some practical data on the efficiency of treating the waste by mechanical filtration. The filter and accessories were loaned by the New York Continental Jewell Filtration Company, of New York, thru the courtesy of Mr. R. E. Milligan, assistant general manager, and the expenses of shipment and installation were borne by the paper company.

Description of the Filter

The filter used is shown in Figure 1. The waste water, after passing the regulating butterfly valve, divided into two streams and entered the bottoms of the two sedimentation tanks. These tanks were of cypress, each having a diameter of 4 feet and a depth to the overflow line of 3.2 feet. The capacity of the two was 603 gallons. The water was delivered to the tanks thru half-inch holes in the sides of risers extending to the surface. The vertical distribution was, therefore, very uniform. Vertical baffles caused the water to flow completely thru the tanks, and a funnel-shaped overflow collected the clarified water from the surface of each tank and delivered it to the surface of the filter. The filter proper was of galvanized iron, set upon a cast-iron base, provided with a false bottom of fine brass screening. The diameter of the filter was 20 inches, and its total depth to the false bottom was 5 feet. Suitable

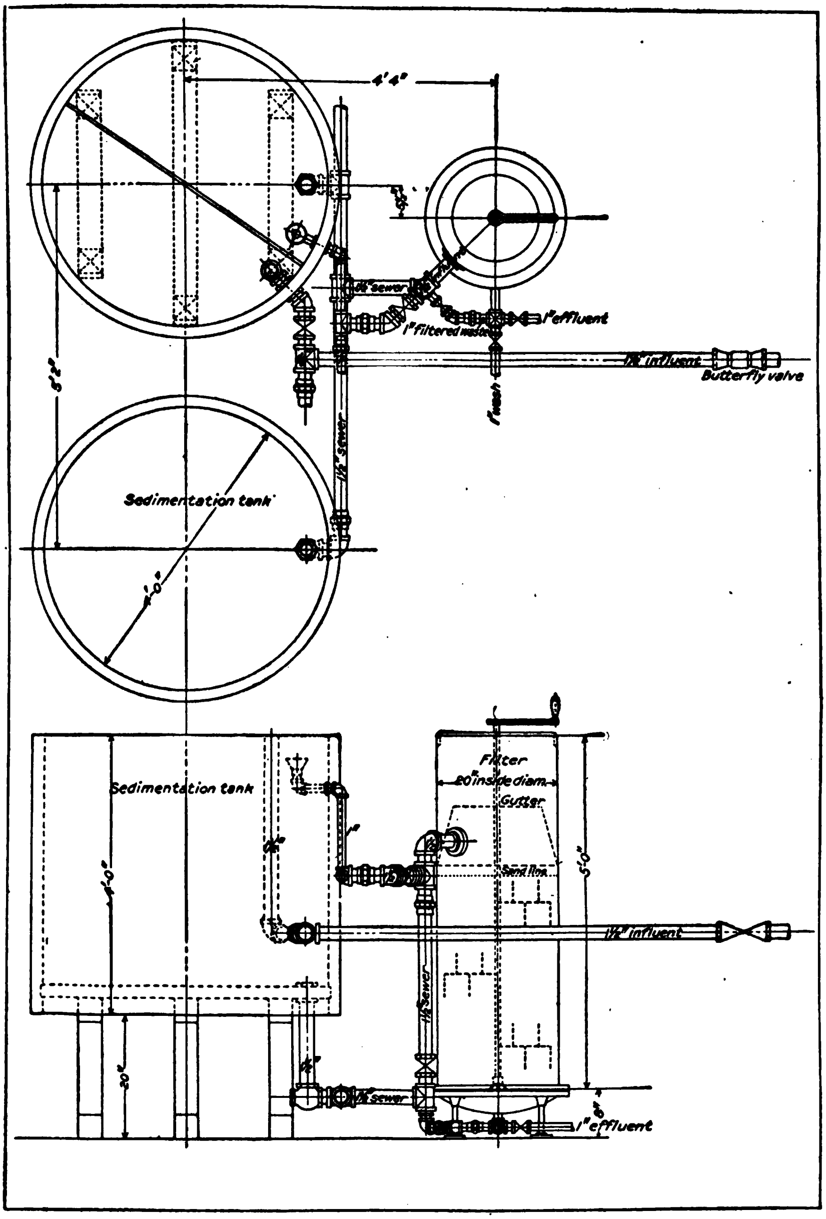


FIG. 1—THE FILTER

connections and valves were provided, as shown in the drawing, so that the influent and the effluent water could be cut off, a stream of wash water could be admitted from the bottom, and the dirty wash water, overflowing from the top, could either be wasted thru the sewer connection or returned to the sedimentation tanks. The device for stirring the sand during washing was operated by hand. Wash water was supplied from an elevated tank under such a head that a maximum flow of 25 gallons per minute, or 12 gallons per square foot per minute, was possible. A flow of about 8 gallons per square foot per minute was generally used. Suitable connections were also made between the bottoms of the sedimentation tanks and the sewer. A 50-gallon barrel at the end of the sewer line served as a measuring tank for the wash water or the sludge. The rate of flow thru the filter was regulated at the outlet by a standard orifice, a sectional view of which is shown in Figure 2. The orifice itself was a trifle over seven-sixteenths of an inch in diameter. The brass cap containing the orifice was screwed over a nipple of 1-inch pipe, which was in turn connected with a 2-inch tee. The opposite side of this tee was connected by a short pipe to the valve regulating the outflow, and a glass tube, fitted to the upper opening of the tee, permitted the static head on the orifice to be read at any time. The large sectional area of the tee as compared with that of the orifice reduced the velocity of approach to a negligible value. This device was roughly calibrated in the field and was subsequently carefully cali-

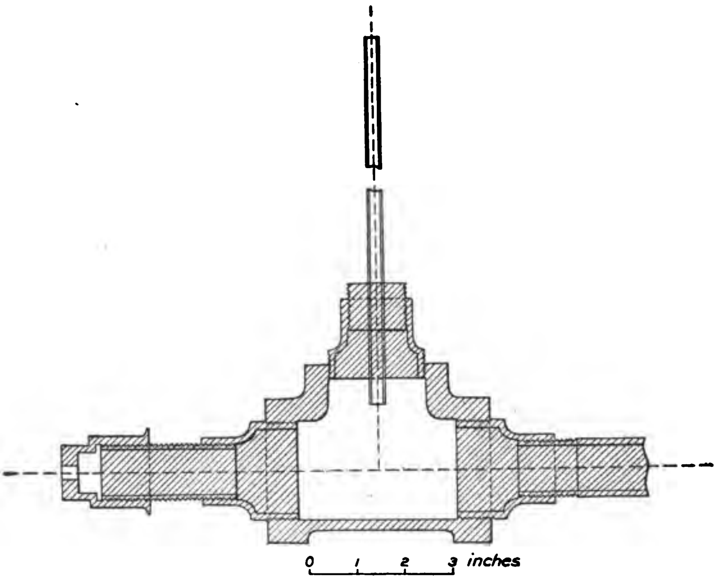


FIG. 2

brated by discharging into a large measuring tank. The rough field values were found to be substantially correct.

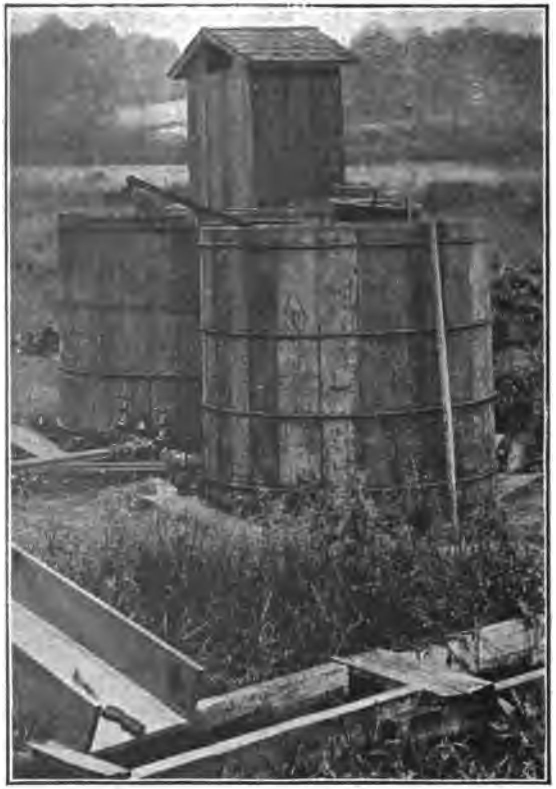
The rate of inflow to the tanks was adjusted to the rate of filtration by the butterfly valve, actuated by a float in one of the tanks (not shown). The filter was filled to a depth of 3 feet with clean 16–30 beach sand, *i.e.*, sand which will all pass a screen with 16 meshes to the linear inch and be retained on a screen with 30 meshes. The water line was 18 inches above the surface of the sand.

The appearance of the filter and sedimentation tanks as they were set up is well shown in Plate II. Plate II, *A,* shows the sedimentation tanks, the influent pipe and controlling valves, and the lever arm of the float operating the butterfly valve. Plate II, *B,* shows the filter itself, the loss-of-head gage, and the connections with the tanks and the sewer. To the extreme right is the measuring tank, and passing between the two tanks on the ground the wash-water pipe line can be seen. The outlet of the filter is hidden from view behind the mound of earth in the foreground.

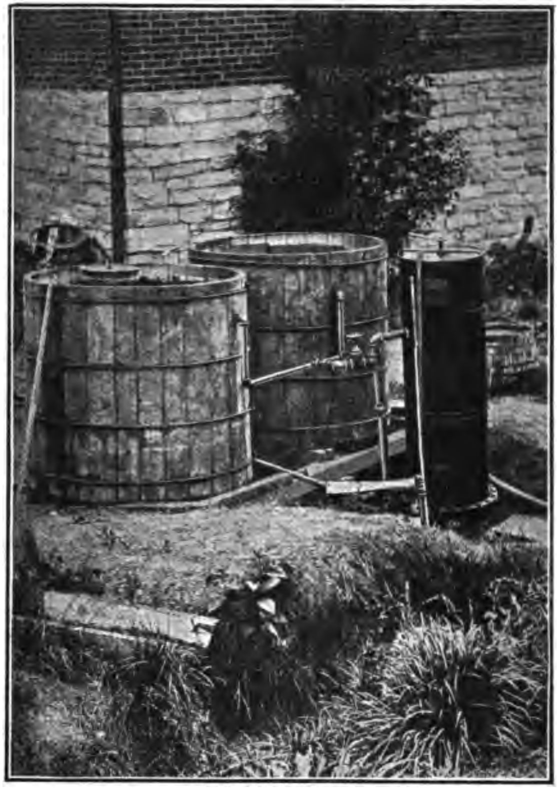
Preliminary Sedimentation

A number of tests were first made to determine the rate of change in the loss of head during filtration with varying periods of preliminary sedimentation. A uniform rate of filtration of 3.25 gallons per minute, equivalent to a rate of 100,000,000 gallons per acre per day, was maintained. The capacity of each of the sedimentation tanks was 300 gallons to the overflow line, so that one tank alone gave a mean storage capacity of one and one-half hours and the two tanks taken together three hours. During the first series of tests both sedimentation tanks were used. The following table shows the relation of loss of head to the quantity of water filtered, the run in question being a fair average of the five made in this way. The results of all the five tests were practically identical.

PLATE II



A. Sedimentation Tanks and Float Regulator



B. Experimental Filter, Sedimentation Tanks, and Measuring Tank

RELATION OF LOSS OF HEAD TO QUANTITY OF WATER FILTERED

Time.	Elapsed time.	Quantity filtered.	Loss of head.
	Minutes.	Gallons.	Inches.
9.35 A.M.	.	. .	4
9.50 A.M.	15	49	7
10.05 A.M.	30	98	8
10.30 A.M.	55	178	12
1.40 P.M.	55	178	11
2.00 P.M.	75	211	14
2.30 P.M.	105	308	20
3.00 P.M.	135	406	28
3.30 P.M.	165	504	40
4.00 P.M.	195	600	55

Between 10.30 A.M. and 1.40 P.M. test shut down. No liquor.

The maximum loss of head possible was 55 inches. The mean results from the above and four similar tests gave 585 gallons of water filtered before the loss of head was total.

One of the sedimentation tanks was then cut out of service and the period of sedimentation correspondingly reduced to one and one-half hours. Four runs to determine the maximum amount which could be past under the conditions gave a mean result of 575 gallons, in practical agreement with that of the first series. It is apparent that a period of one and one-half hours was sufficient for the removal of that portion of the suspended matter which can be removed by sedimentation. An examination of the water during sedimentation in a glass vessel tended to confirm this conclusion. The suspended matter appears to be of two distinct kinds. One is very finely divided and light and does not settle out even on prolonged sedimentation. The other is granular and heavy and settles rapidly and completely. It was not convenient with the apparatus at hand to reduce further the period of sedimentation. The laboratory experiments quoted earlier in this paper indicated that a period of one hour would accomplish practically the same result as more prolonged sedimentation.

Amount of Wash Water

The pressure on the wash-water pipe line was sufficient to give an upward flow thru the filter of 25 gallons per minute, or about 12 gallons per minute per square foot of surface. In practise, flows of 8 to 10 gallons per square foot per minute were used. After each of the test runs just described, the filter was washt with an amount of water varying from 50 to 100 gallons for two and one-half to five minutes, the wash water being carefully measured. Observations of the surface of the filter showed plainly that the shorter period of washing was sufficient and that the longer periods did not in any case increase the quantity filtered during the succeeding run. The material deposited on the surface was exceedingly light and finely divided, and required but little water for its removal. The stirring device used was not well adapted to the purpose, since there was little penetration of the sand by the suspended material. An air blast or some device which would thoroly agitate the surface before washing would undoubtedly make possible the use of still less wash water. Allowing 50 gallons for each wash, with an average quantity of 575 gallons per run filtered, gives a little less than 9 per cent. wash water.

The loss of head in these tests bears out a fact of general observation, that the loss of head increases at a fairly constant rate during the run. Such being the case, it is quite evident that with a deeper filter the quantity filtered between washings would be proportionately increased. The amount of wash water would, on the other hand, remain constant or increase but slightly. It is therefore safe to say that with a filter having an available loss of head of 9 to 12 feet an average run of six hours or more could be maintained and the amount of wash water could be reduced to less than 5 per cent.

Disposal of Wash Water

The process, as thus far described, is merely one of concentration of the suspended matter. If the above estimate of 5 per cent. of wash water be correct and the filtered water be used for washing, the net result of the process would be the separation of 95 per cent. of the water in comparatively clean form, leaving the total suspended impurity in the remaining 5 per cent. of the water. This water might be spread upon a small area of land to evaporate, it might be evaporated by artificial means, or, if preferable, it might be filter prest; but the most economical way to dispose of it is to return it directly to the sedimentation tanks. The material on the filter is so prest together into small lumps that it settles rapidly. The problem of the ultimate disposal of the sludge from the tanks will be taken up in another place.

Character of Effluent

Complete chemical analyses of the waters before and after treatment during these experiments were neither desirable nor feasible. The determination of the total organic solids contained in the waters is the only analytical determination which would be of service in estimating the efficiency of the process. This determination might have been made by the usual indirect method, *i. e.*, by the difference between the total and the fixt or mineral solids. That determination is, however, not well adapted to field conditions, and small changes in the organic content would be entirely masked by the relatively large amount of mineral residue present. It was found in the laboratory studies on the waste liquor that estimating the efficiency of the various precipitation methods on the basis of this organic-solids determination, and also on the basis of the oxygen-consumed method, gave practically the same results. In other words, the organic matter present is fairly homogeneous in character, as might well be expected, and the oxygen-consumed figures, altho purely relative, are therefore a reliable measure of the amount of organic matter present. This test, therefore, and the determination of the turbidity, or suspended solids, by the well-known Jackson turbidimeter method, were used in this work to determine the quantity of the organic and the suspended matter, respectively.

During the two three-hour runs samples for analysis were collected every ten minutes from the raw liquor, sedimented liquor, and final effluent, the small samples being mixt together to obtain an average sample for the run. In both cases the rate was maintained at 3.25 gallons per minute. The following data of the test were obtained:

DATA FROM TEST RUN OF FILTER

Test number.	Duration.	LOSS OF HEAD.		Quantity filtered.
		Initial.	Final.	
	Hours.	Inches.	Inches.	Gallons.
7	2.9	4	55	566
8	3.0	3	55	585

The chemical analyses for the two sets were very similar. The following table shows the mean values obtained:

ANALYSES OF RAW LIQUOR, SETTLED LIQUOR, AND EFFLUENT

	Parts per million.	Per cent. removed.
Raw liquor.		
Oxygen consumed:		
Total	9,400	..
Dissolved	2,000	..
Suspended	7,400	..
Suspended solids	28,000	..
Settled liquor.		
Oxygen consumed:		
Total	6,200	34
Dissolved	1,600	20
Suspended	4,600	38
Suspended solids	4,400	84
Effluent.		
Oxygen consumed:		
Total	2,200	77
Dissolved	1,700	15
Suspended	500	93
Suspended solids	575	98

The appearance of the effluent frequently changed materially during the run. Contrary to expectations and general experience, the best effluent was always obtained during the early part of the run. The very first portions of effluent were generally slightly turbid, after which a perfectly clear water came thru, as a rule, entirely free from suspended matter. This state of affairs continued until the loss of head had reached about 6 or 8 inches, when the effluent suddenly became turbid and continued so uniformly until the end. The inference was that under a tension or pull on the surface film represented by about 6 to 8 inches of head the finely divided matter contained in the liquor was pulled thru the surface and then thru the filter. It was thought that if this should be a correct interpretation of the phenomenon, it might be obviated in one of two ways—by increasing the toughness of the surface deposit, or by decreasing the size of the sand pores by use of a layer of finer material on the surface. Both of these methods were tried.

To increase the density and toughness of the surface film the surface was covered with liquor, the outlet being closed, and 2 ounces of sulfate of aluminum were added and stirred in. Fifteen minutes were allowed for the formation and settling of the precipitate, and the filter was then started. The following data were obtained.

DATA OF TEST RUN OF FILTER AFTER ADDITION OF ALUM

Duration of run	**1.9**	**hours**
Loss of head:		
Initial	**14**	**inches**
Final	**55**	**inches**
Quantity filtered	**370**	**gallons**

The chemical analyses resulted as follows:

ANALYSES OF RAW LIQUOR, OF SETTLED LIQUOR, AND OF EFFLUENT AFTER THE ADDITION OF ALUM

	Parts per million.	Per cent. removed.
Raw liquor.		
Oxygen consumed:		
Total	9,000	..
Dissolved	1,600	..
Suspended	7,400	..
Suspended solids	27,300	..
Settled liquor.		
Oxygen consumed:		
Total	6,000	33
Dissolved	1,600	6
Suspended	4,400	40
Suspended solids	4,500	84
Effluent.		
Oxygen consumed:		
Total	1,800	80
Dissolved	1,600	..
Suspended	200	98
Suspended solids	450	98

The improvement of this effluent over that obtained without a coagulant is decided, but is hardly sufficient to warrant the extra cost of treatment. Previous studies had shown that it was necessary to add to the volume of waste on the surface of the filter at least 2 ounces of alum in order to bring about a satisfactory precipitation of the lime. Any less amount would not have produced a satisfactory precipitation in a short time. The total quantity filtered during the run was 370 gallons. This gives an amount of sulfate equal to a little over 2 grains per gallon for the entire run, or a consumption of 300 pounds of alum per million gallons. At a cost of 1 cent per pound (based on net weight of sulfate of aluminum in the crude compound) this would amount to $3 per million gallons of waste, or 10 cents per ton of finished board. This additional cost for chemicals, together with the slightly additional expenditure for labor and water required by more frequent washing, would not seem to be justified in general by the slightly increased purity of the effluent. The results are given here and the cost estimated as a matter of record and in order to show what could be accomplished by the process if it were deemed desirable in any special case.

The second plan for preventing the small suspended particles in the liquor being drawn thru the surface was to use a surface layer of fine sand. A suitable grade of sand was obtained locally and screened thru a No. 20 screen. The sand thus prepared contained a considerable amount of rather fine material. Mechanical analysis showed that it had an effective size of 0.26 mm. and a uniformity coefficient of 2.0.

A 6-inch layer of the sand was placed on the top of the filter and washt. After washing, the surface was examined and found to consist of a well-mixt layer of coarse and fine material. Two runs were made with the filter as thus prepared. The following are the average data obtained:

DATA OF TEST RUN OF FILTER AFTER ADDITION OF FINE SAND

Duration of run	2.3	hours
Loss of head:		
Initial	6	inches
Final	55	inches
Quantity filtered	449	gallons

The appearance of these effluents was such that chemical analyses were not made. They were essentially like the others, and there was no advantage in the use of the layer of fine sand which could in any way offset the obvious disadvantage of the much shorter run.

The Sludge

Amount and Composition.—Thruout the experiments, except for the first few runs, the dirty wash water from the filter was run back into the sedimentation tanks. Eventually, therefore, all the material removed from the waste liquor was accumulated in the bottom of these tanks. At the conclusion of the experiments, which consisted altogether of 12 runs, dealing with a total of 4,900 gallons of the waste, the liquor was drawn off from the tanks as far as possible and the depth of accumulated sludge and remaining liquor was measured. The average depth was found to be 6 inches. After thoro mixing, a sample of the sludge was collected and allowed to stand in a glass cylinder for two hours. From a column of sludge 12 inches in depth a 2-inch layer of clear liquid separated at the top, leaving the depth of sludge on the bottom 10 inches. Therefore the 6 inches of sludge and liquor left in the tank was composed of 5 inches of sludge and 1 inch of water, so that 5 inches represents very closely the average depth of sludge in each tank and will be used in the calculations which follow.

Analysis of the sludge gave the following results:

ANALYSIS OF SLUDGE RESULTING FROM FILTRATION OF STRAWBOARD WASTE

Specific gravity	1.094	
Total solids	19.4	per cent.
Volatile	9.8	per cent.
Fixt	9.6	per cent.

From these data the following results are obtained:

Sludge in tank	78	gallons
Liquor treated	4,900	gallons
Per cent. of sludge (by volume)	1.6	

From 1,000,000 gallons of waste there will be obtained:

Sludge	79	cubic yards
Dry solids	28,000	pounds
Organic solids	14,100	pounds
Mineral solids	13,900	pounds

For each ton of strawboard there will be produced:

Sludge	4.7	cubic yards
Dry solids	1,680	pounds

A glance at Plate I, *B*, will indicate what these figures mean. The piles of straw there shown represent three months' supply for a mill producing 30 tons of board per day. Over one-third of the total weight of this straw will appear in the waste liquor.

Properties and Uses of the Sludge.—As is indicated by the analysis, the sludge is a thick mud. It is dark, muddy brown in color, and has the odor of straw. On exposure to the air it dries much more rapidly than would be expected, owing largely to the action of the air on the free lime. If allowed to dry in a mold, the resulting cast comes away clear and has taken a very clean impression of the mold. It is very brittle by itself. If, however, a little paper fiber be mixt with it before drying, the dry cast is fairly tough. Such a mixture could undoubtedly be used in many places as a cheap substitute for *papier-maché.*

The dry product had a specific gravity of 0.62. It will readily absorb considerably more than its own weight of water. Its porous nature suggests its use as a heat insulator, particularly in refrigerators.

This material possesses a certain value as a fertilizer. It contains a considerable proportion of calcium carbonate, a substance beneficial to cultivated land. It also contains 0.34 per cent. of phosphoric acid in an available form. Its loose, porous nature makes it particularly valuable for use on clay land, as such land is made much more porous by it and at the same time is sufficiently fine to retain the moisture well. This point was well brought out on examining one of the abandoned settling basins. This basin had originally been diked up about 6 feet above the surrounding fields, and had subsequently been completely filled with the sludge. On digging into this deposit, standing water was observed at a depth of less than 2 feet below the surface, altho the ground water level in the adjacent fields was at least 6 feet lower and the field had not been flooded for over a year.

Some experiments have been made with this solid residue as a base for the production of a sulfur dye. Fusion of the substance with twice its weight of a mixture of one part of sulfur and two of sodium sulfide yields a mass containing a considerable proportion, perhaps 20 per cent., of a brown sulfur dye. Purification of the resulting dye from the mass of inert material was not attempted, and further study is required to determine the most satisfactory procedure for the preparation and purification of this dye, but enough has been done to indicate that a low-grade sulfur dye can be made from this waste material. It dyes unmordanted cotton a good warm shade of brown, is fast to hot soap and to sunlight, and compares favorably in color and fastness with dyes on the market selling at 15 cents per pound.

Considerable attention has been given to the problem of finding profitable use for the residual organic matter which is the end product of the filtration processes here recommended. Altho the satisfactory solution of this problem would be highly desirable and would at once assure the treatment of all strawboard waste liquors, it must be remembered that this is merely a question incidental to the main problem. It is believed that the process of sedimentation, followed by mechanical filtration, essentially as described, furnishes by itself a solution of that problem which will prove of financial advantage to the manufacturers and of lasting benefit to all who have an interest in the purity of the streams below. It was therefore not deemed pertinent to the present investigation to pursue further the question of the utilization of the sludge. The suggestions here given, and particularly that in regard to the preparation of a dye, are merely intended to indicate the direction in which, according to the writer's views, future research would bring results of value.

Résumé of Method of Treatment

The following plan of treatment is recommended: The waste water from the beaters should be past thru a properly designed settling tank of such size that it will give a period of about one hour's storage. A tank built on the principle of the Dortmund tank would be preferable to any other, owing to its compactness and the possibility of continuous use. The tank should have a capacity of approximately 1,200 gallons for each ton of board produced per day by the mill.

The water should pass from the tank directly to the filters, which should be built in small units. The filters should have an available loss of head of at least 12 feet, and could be run at a rate of 100,000,000 gallons per acre per day. This would necessitate a filter area of approximately 16 square feet per ton of board produced daily. This figure allows 10 per cent. excess for repairs and washing.

The effluent should be pumped and used again for washing the filter and for beater wash water. It would at all times contain less impurity than does the very last water past from the beaters, and its use during the early part of the washing process could do no possible harm. To insure a proper washing out of the last of the lime from the stock, it would probably be found necessary to complete the washing with fresh water. In any case a saving of one-half to two-thirds of the amount of wash water now used could be realized, the amount of waste liquor being correspondingly reduced. The water used in washing the filter should be returned to the sedimentation tank, and would require no further treatment.

The sludge from the sedimentation tank could be most expeditiously dealt with by filter pressing. There would remain a fairly dry and hard press cake, which would require disposal unless some profitable use for it be found. Under the influence of the air and sun this material is speedily resolved into mineral matter, forming a sort of soil. It could be spread upon low land in thin layers and would then take care of itself.

Instead of being filter prest, the sludge might be discharged upon small areas of sandy soil where such are available or even upon somewhat larger areas of less porous soil, and allowed to dry out by draining and evaporation. The dry layer of sludge could then be removed and disposed of. No accumulation of the wet material should be allowed. When once dried out and subjected to the action of the sunlight and air it soon becomes harmless, but in deep beds and constantly covered with water it is subject to all the offensive putrefaction described in the case of the sedimentation fields. This process of spontaneous draining and drying out would probably be found the cheaper, but would require a considerable area of suitable land and would also involve more careful supervision. The amount of land required would depend wholly on the quality of the drainage. For average conditions it may be roughly estimated at 1,000 square feet per ton of board produced by the mill daily. The dried sludge would require removal before the application of more sludge.

Character of Waste Water after Treatment

The quality of the effluent from the proposed process is not all that could be desired. In spite of the fact that it is possible to remove over 90 per cent. of the suspended organic matter present in the liquor, the resulting effluent is still putrescible. That is, a bottle of the liquor shut up air-tight and kept at a warm temperature will in the course of a few days undergo putrefactive fermentation. The putrefaction is rather slow, however, and not offensive. In admixture with an equal volume of tap water the effluent does not putrefy even when inoculated with a small amount of sewage.

The fact that the effluent is practically free from gross suspended solids—that is, solids which will settle out on the bed of the stream—is of greater significance than the analytical data. As was pointed out in the earlier discussion of the problem (page 8), by far the greater part of the damage done to the polluted streams is brought about by the deposit of material in the stream beds. The effluent which has past the treatment here outlined will contain no material capable of settling out even in slack water.

The conclusion is therefore justified that the discharge of this effluent into a stream whose minimum flow equals in volume the flow from the mill would not produce a nuisance. With no allowance for a possible diminution of the amount of wash water, if used as suggested above, this means that the minimum stream flow must be at least 0.05 cubic foot per second per ton of board produced daily at the mill. A minimum flow of 0.1 cubic foot per second per daily ton would therefore provide for a generous factor of safety if the stream is reasonably unpolluted at the point of discharge.

The discharge of such an effluent into a stream highly polluted with sewage would probably be accompanied by an aggravation of the conditions already existing. There would, as usual, be a precipitation of the sewage material by the lime of the effluent. In all fairness it could be maintained that the organic matter of the effluent was not contributing to the nuisance. If, therefore, it should be decided that the addition of lime to a sewage polluted stream constitutes a nuisance, further treatment of the effluent would be required to remove this caustic lime. If, on the other hand, as would seem to be reasonable, it should appear that the real pollution of the stream lay in the sewage and the lime was incidental and was there by right, there would be no reason for such further treatment.

Cost of the Process

The cost of the process recommended cannot be discust in detail at this time. It would be largely the cost of installation. The design and construction of a suitable plant require expert attention, and the cost of the plant would depend somewhat on local conditions. The cost of maintenance would be slight. The cost of additional pumping to supply wash water—say 5 per cent. more than the present amount pumped—can be determined readily. The services of one man, who could also assist in the mill part of the time, would be a sufficient addition for running the plant. The power necessary to pump the sludge thru the filter presses, if that plan were adopted, would not be noticeable in comparison with the total power used in the mill. If the sludge were disposed of by the other plan, evaporation and drying in the field, the services of another man would be required.

In conclusion, the following summary of the chief points of the present paper may be made:

SUMMARY

The present method employed for the treatment of strawboard waste liquor, that of sedimentation in large fields, is unsatisfactory and expensive and proceeds on a wrong principle.

The filtration of the liquor thru sand, so-called mechanical filtration, but without coagulants, after a short period of sedimentation will remove 93 per cent. of the suspended organic solids and 98 per cent. of the total suspended matter determined as turbidity.

Actual experiments have shown that the resulting effluent can be discharged into an equal volume of reasonably pure water without creating a nuisance. In particular, there will be no sedimentation of organic matter. If its discharge into a seriously polluted stream should bring about precipitation of sewage matter and consequent nuisance, the blame should in all equity be fixt on those contributing the sewage.

The sludge resulting from the sedimentation tanks, after pressing or spontaneous drying, is innocuous and makes good soil. It has some value as a fertilizer and is particularly valuable to mix with the clay soils of the Middle West to render them more porous. Other uses for this material are suggested, but they require further study.

The cost of the entire process is sufficiently low to admit of its adoption wherever stream pollution is serious.

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CONTRIBUTIONS FROM THE
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THE DETERMINATION OF THE ORGANIC NITROGEN IN SEWAGE BY THE KJELDAHL PROCESS

II. STUDIES ON DIRECT NESSLERIZATION

BY LEYLAND WHIPPLE

THE value of a method of analysis, when used in routine work, becomes to a considerable extent a function of the rapidity with which it may be carried out. In no case, perhaps, is this better exemplified than in the estimation of organic nitrogen by the Kjeldahl process, in sanitary water and sewage analysis. This determination, by far the most important, from the viewpoint of interpretation, involves one of the longest procedures, and is consequently not generally employed.

Several attempts have been made to shorten this process by nesslerizing the diluted digestate directly, without distillation. Unless special precautions are taken, however, certain constituents of the digestate are found to react with the Nessler's reagent, producing a turbidity in the presence of which it is not possible to make close comparisons of the color of the nesslerized tube with that of the standard. Of the heretofore published methods, only one, that of Kimberly and Roberts (1906), has been found to be uniformly successful under local conditions, and this method has in our hands failed to yield satisfactory results.

It is the purpose of this paper to present the results of some experiments carried out in this laboratory on direct reading of Kjeldahl digestates, and to give a procedure which has been found entirely successful here and which may be of application elsewhere.

PREVIOUS RESULTS

Direct reading of diluted Kjeldahl digestates has been the subject of investigation both in this country and abroad. Rideal (1901) gives a procedure which consists in neutralizing with an excess of caustic soda, diluting to a definite volume, allowing to settle for a number of hours, and pipetting off a portion of the clear supernatant liquid for nesslerization.

Phelps (1904), at this laboratory, tried making up the digested sample to volume, taking out the requisite amount, neutralizing and making up to 50 c.c. in the Nessler tube. He found that in about half the cases a turbidity was produced on adding the Nessler reagent.

McGowan (1904) attempted to use a modification of the Rideal method by making *just neutral* with hydrate, and in some cases adding potassium oxalate to precipitate the calcium. He found, also, that in more than half the total number of determinations the tubes were too turbid to read. Furthermore, he pointed out a probable loss of ammonia by adsorption in the precipitate formed. He attributed the turbidity to the insufficient precipitation of the calcium.

As a result of an extended investigation carried out at the Columbus Sewage Testing Station, Kimberly and Roberts (1906) published a modification of the processes of Rideal and McGowan, which consisted in diluting the digestate to 100 c.c., nearly neutralizing with sodium hydrate, cooling, adding a slight excess of hydrate and then 2 c.c. of a 10 per cent. solution of sodium carbonate solution, diluting to a known volume, allowing to settle six hours, and nesslerizing a portion of the clear liquid. This method was the result of a series of investigations into the causes of turbidity, the latter being finally attributed to the presence of calcium and magnesium salts. Kimberly showed that an amount of calcium up to 2.5 mg. per nesslerized tube would not produce turbidity, but that in greater concentrations it would interfere with the process. He used a slight excess of caustic to precipitate the magnesium, and sodium carbonate to precipitate calcium. The long period of settling was necessary in order to remove the slow-forming calcium carbonate.

He states, further, that potassium permanganate cannot be used to complete the digestion, as the manganous sulphate formed will cause turbidity on nesslerizing. The cooling before final neutralization was recommended to avoid the escape of ammonium hydrate from the hot solution. He found it necessary, also, to purify the soda by permanganate, in order to remove iron and organic impurities. In his experiments on the accuracy of the process, he found that adsorption was a negligible factor. In numerous comparative determinations by distillation and direct nesslerization close agreements were obtained.

In an endeavor to obtain a method for the direct reading of Kjeldahl digestates at this laboratory, the method of Kimberly and Roberts was tried, but failed to yield satisfactory results, it being found impossible to obtain clear tubes upon nesslerization. We have therefore further investigated the causes tending to produce turbidity, and have succeeded in devising a modification of the Rideal method which is successful under conditions found here, and which should be applicable to all sewages except under conditions which will be defined later on in this paper.

CAUSES OF TURBIDITY IN THE TUBE

The causes of the turbid tube have been variously attributed to the presence of calcium, magnesium, sulphate ions, and manganous salts, resulting from the use of potassium permanganate to complete the digestion.

Kimberly and Roberts have shown that sulphate ions are not one of the disturbing causes. In regard to manganous salts, it is a well-known fact that permanganate is rapidly hydrolyzed and reduced in dilute solutions, either weakly acid or alkaline, and is then precipitated on standing. When used to complete the digestion, the permanganate is at once reduced by the hot acid, and settles out of the diluted digestate as a basic salt. Moreover, should traces of manganous sulphate yet remain in solution, they will be precipitated upon neutralization. Permanganate has been used throughout the present investigation and has never given rise to any cloudiness in the tube.

We have confirmed the conclusion of Kimberly and Roberts that calcium may be present in the tube up to 2.5 mg. without producing turbidity. As an amount of sewage is normally taken which will give a reading of not more than five on the colorimetric standards, corresponding to .05 mg. of nitrogen, it is clear that we cannot have more than fifty parts of calcium to one of nitrogen. That is, in a sewage in which the calcium content is not over fifty times that of the nitrogen, tubes which will give a reading of five or less may be nesslerized directly without any special precautions being taken to remove the calcium. Except in the case of a good sand-filter effluent from a sewage very high in calcium, this ratio will seldom be realized. Even in cases where this ratio is exceeded, it might be advisable to use a less amount of the sample, so as to make the reading about two on the standards. This would increase the allowable ratio of calcium to nitrogen to 125.

Similarly the allowable ratio of magnesium to nitrogen was determined, and found to be so low that the removal of magnesium appears to be an essential detail. An amount of magnesium in the tube of over .025 mg. will produce either a cloudiness or a considerable deepening of the color of the reagent. To determine the amount of excess hydrate required to effect sufficient precipitation of magnesium salts, a sample was made up containing about 0.12 mg. nitrogen as ammonium chloride, 2.5 mg. magnesium as chloride, and digested with 5 c.c. of sulphuric acid in the usual way, with the addition of permanganate. The digestate was diluted to 250 c.c., and successive portions of 10 c.c. each were neutralized with varying amounts of a 5 per cent. solution of sodium hydrate, starting with an amount just sufficient to neutralize the acid, as determined by a test portion, using phenolphthalein as an indicator. The samples were filtered through washed filters into Nessler tubes, and the filter and precipitate then washed with ammonia free water.

The results of this experiment are shown in the following table:

Tube.	5 per cent. hydrate. c c.	Reading.	Remark.	Corrected reading.*
1	7.2	. .	Very turbid	. .
2	7.5	. .	Very turbid	. .
3	8.0	About 5.0	Considerably turbid	4.8
4	9.0	5.0	Slightly turbid	4.8
5	10.0	4.8	Clear	4.6
6	12.0	4.9	Clear	4.7
7	15.0	5.0	Very slightly turbid	4.7
8	20.0	About 5.8	Considerably turbid	5.4

* A blank on 10 c.c. of the hydrate gave a reading of 0.2; the figures in this column have had this correction applied.

This indicates that after neutralization an excess of hydrate must be added equivalent to 1.5 grams per 100 c.c. of the diluted digestate, in order to effect a satisfactory removal of the magnesium. Furthermore, it is indicated, and we have further confirmed the fact, that the reading is not appreciably affected by an excess of hydrate unless in sufficient amount to itself cause turbidity. This amount has been found to be about 0.35 gram per 50 c.c. Nessler tube.

The addition of copper sulphate to hasten the digestion has no effect on the final result, as the copper is entirely removed by the hydrate.

It appears, therefore, that magnesium salts are one of the principal causes of turbid Nessler tubes; that the magnesium can be satisfactorily removed by an excess of sodium hydrate; and that this necessary excess is not in itself sufficient to produce turbidity.

METHOD

The detailed procedure which has been in use at this laboratory for the past three months is as follows:

Reagents.—The same as those used in ordinary Kjeldahl determinations of organic nitrogen, with the addition of a 5 per cent. solution of potassium or sodium hydrate.

Procedure.—Fifty c.c. of the sample (or a greater or less amount, according to the nitrogen content) are digested with 5 c.c. of concentrated sulphuric acid, nitrogen free, and a small amount of copper sulphate until colorless. The digestion is completed by the addition of a few small crystals of permanganate to the hot acid. After cooling, the digestate is diluted to 250 c.c., mixed, and a portion (say 25 c.c.) is pipetted out, mixed with an equal portion of 5 per cent. sodium hydrate solution, and filtered through a filter paper which has been washed previously with 100 c.c. or so of ammonia free water to remove traces of ammonia. Twenty c.c. of the filtrate are pipetted into a Nessler tube, made up to 50 c.c. with best water, mixed by pouring from one tube to another, nesslerized, and read after ten minutes. A blank should be run on the entire process, and the result subtracted from the reading.

DISCUSSION

This method has been in use in this laboratory for the past three months, and has given uniformly satisfactory results. In no case have we experienced any difficulty in getting a clear tube of the correct color to match the standards, even in tubes reading as high as seven or eight on the standards, corresponding to .07 and .08 mg. of nitrogen in the tube. In practice, however, readings of over five are discarded and new dilutions made, the rapidity of the process making it readily possible to introduce this desirable procedure.

In all these experiments, and in our regular work, the ordinary purified caustic soda in sticks has been used, preliminary purification having been found unnecessary.